THE

ACTUAL LATERAL PRESSURE

OF

EARTHWORK.

BY

BENJAMIN BAKER, M. Inst. C.E

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PREFACE.

THE much discussed subject of the pressure of earthwork is in this essay so exhaustively treated that nothing is left to be desired. The engineer may find here a satisfactory explanation of the causes of the discrepancies between theory and practice, and of the differences between different authorities.

It was originally presented as a paper to the Institution of Civil Engineers, from the published minutes of which the essay as here presented was published in VAN NOSTRAND'S MAGAZINE. Abstracts only of the discussion are presented here.

The Actual Lateral Pressure of Earthwork.

THE fact that a mass of earthwork tends to assume a definite slope, and that if this tendency be resisted by a wall or any other retaining structure, a lateral pressure of notable severity will be exerted by the earthwork on that structure, must have enforced itself upon the attention of constructors in the earliest ages. Many of the rudest fortresses doubtless had revetments, and of the hundreds of topes, or sacred mounds, raised in India and Afghanistan two thousand years ago, not a few afford examples of surcharged retaining walls on as large a scale as those occurring in modern railway practice. Nevertheless, long as the subject has occupied the attention of constructors, there is probably none other regarding which there exists the same lack of exact experimental

data, and the same apparent indifference as to supplying this want. Thousands of pieces of wood have been broken in all parts of the world to determine the transverse strength of timber, whilst the experiments that have been undertaken to ascertain the actual lateral pressure of Earthwork are hardly worth enumerating. One authority after another has simply evaded the task of experimental investigation, by assuming that some of the elements affecting the stability of earthwork are so uncertain in their operation as to justify their rejection, and have so relieved themselves from further trouble. It would hardly be less logical to assume that because timber is liable to become rotten and possesses no strength at all, it was therefore unnecessary to conduct experiments in that case also. As a matter of fact, although these uncertain elements are neglected in investigations, engineers in designing, and still more contractors in executing, works, do not neglect them, nor could they do so without leading to a blameworthy waste of

money in some instances, and to a discreditable failure in others. The result of the present want of experimental data is then simply that individual judgment has to be exercised in each instance, without that aid from careful experimental investigation which in these times is enjoyed in almost every other branch of engineering.

The mass of existent literature on the subject is both misleading and disappointing, for with little exception the bulk of it consists merely of arithmetical changes rung upon a century-old theory, which even at the time of its inception was put forward but as a provisional approximation of the truth, pending the acquirement of the necessary data. Writing some fifty years ago Professor Barlow excused his "very imperfect sketch of the theory of revetments, at least as relates to its practical application," on the ground that there was a "want of the proper experimental data;" and but comparatively the other day Professor Rankine had to write in almost

identical terms: "There is a mathematical theory of the combined action of friction and adhesion in earth; but for want of precise experimental data its practical utility is doubtful." It is not, therefore, for want of asking that the missing data are not forthcoming. Indeed, the present desiderata could not have been more clearly formulated than they were half a century ago by Professor Barlow in the following words: "To render the theory complete, with respect to its practical application, it is necessary to institute a course of experiments upon a large scale; upon the force with which different soils tend to slide down when erected into the form of banks. A well-conducted set of experiments of this kind would blend into one what many writers have divided into several distinct data. Thus some authors have considered first, what they call the natural slope of different soils, by which they mean the slope that the surface will assume when thrown loosely in a heap; very different, as they sup-

pose, from the slope that a bank will assume that has been supported, but of which that support has been removed or overthrown. This, therefore, leads to the consideration of the friction and cohesion of soils, and what is denominated the slope of maximum thrust; but, however well this may answer the purpose of making a display of analytical transformations, I cannot think it is at all calculated to obtain any useful practical results. I should conceive that a set of experiments, made upon the absolute thrust of different soils, which would include or blend all these data in one general result, would be much more useful, as furnishing less causes of error, and rendering the dependent computations much more simple and intelligible to those who are commonly interested in such deductions."

A knowledge, however imperfect, of the actual lateral pressure of earthwork, as distinguished from what may be termed the "text-book" pressures, which, with hardly an exception known

to the author, are based upon calculations that disregard the most vital elements existent in fact, is of the utmost importance to the engineer and contractor. It affects not merely the stability of retaining walls, but the strength of tunnel linings, the timbering of shafts, headings, tunnels, deep trenches for retaining walls, and many other works of every-day practice. The vast divergence between fact and theory has perhaps impressed itself with peculiar force upon the author, because, having had the privilege of being associated with Mr. Fowler, Past President of the Inst. C.E., during the whole period of the construction of the "underground" system of railways, he has had the advantage of the experience gained in constructing about 9 miles of retaining walls, and, in relation to the subject of the present paper, the still more valuable experience of 34 miles of deep-timbered trenches for retaining walls, sewers, covered ways, and other structures. A timber waling is a sort of spring, rough it may be, but

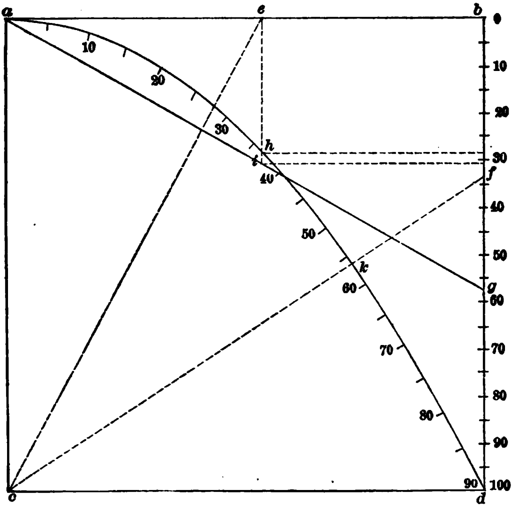
still the deflections when taken over a sufficiently large number of walings afford an approximate indication of the pressure sustained—an advantage which a retaining wall does not possess. Again, though numberless retaining walls have failed, in ninety-nine cases out of hundred the failures have been due to faulty foundations, and, consequently, experiences of this sort seldom afford any direct evidence as to the actual lateral pressure of earthwork. In timbered trenches, on the other hand, the element of sinking and sliding foundations does not so frequently arise to complicate the investigation.

All kinds of earth were traversed by the above 34 miles of trenches, from light vegetable refuse to the semi-fluid yellow clay, which at different times has crushed in so many tunnel linings in the northern districts of the metropolis. The heights of the retaining walls ranged up to 45 feet, the depths of the timbered trenches to 54 feet, and the ground at the back of the former was in many

cases loaded with buildings ranging up to 80 feet in height. Possibly some of the author's observations and conclusions in connection with these and other works of a similar character may be of interest to engineers, though the information he is able to contribute, having been obtained chiefly in the ordinary routine of his practice and not in specially devised investigations, must necessarily form but a very imperfect contribution to the data which have been asked for so long.

The theory underlying all the multitudinous published tables of required thickness for retaining walls is, that the lateral pressure exerted by a bank of earth with a horizontal top is simply that due to the wedge-shaped mass, included between the vertical back of the wall and a line bisecting the angle between the vertical and the slope of repose of the material. If this were true in practice, all such problems could be solved by merely drawing a line on the annexed diagram, in which *a b d c* is a square, *a b g* a triangle, having the sides of the

ratio of 1: $\sqrt{\frac{1}{3}}$, and $a\ h\ d$ a parabolic curve.*



*For earthwork and masonry of the same weight per cubic foot the equation for stability is:

$$\frac{ht^2}{2}=\frac{h^3}{6}\tan^2 \tfrac{1}{2} \text{ angle.}$$

Hence, the required thickness (t_1) in terms of the height (h) will be $t_1=\sqrt{\tfrac{1}{3}}\tan \tfrac{1}{2}$ angle, which is represented on the diagram by the line a–g; and the "equivalent fluid pressure" in terms of that of a cubic foot earthwork will be $=\tan^2 \tfrac{1}{2}$ angle, which is represented by the parabolic curve $a\ h\ d$.

Thus, if it were required to know the lateral pressure per square foot of earth-work, having a slope of repose of $1\frac{1}{2}$: 1, and the thickness of rectangular vertical wall which, when turning over on its outside edge would just balance that pressure, it would merely be necessary to draw the line $c\ f$ at the given slope of $1\frac{1}{2}$: 1 and the line $c\ e$ bisecting the angle $a\ c\ k$, when the line $e\ h$ would give the equivalent fluid pressure in terms of that of a cubic foot of the earth=28.7 per cent., and the line $e\ i$ the thickness of the rectangular wall in terms of the height=31 per cent, the weight of masonry being the same as that of the earth.

Common stocks in mortar and ballast backing each weigh about 100 lbs. per cubic foot, hence, on the preceding hypothesis, the pressure acting on the wall would be the same as that due to a fluid weighing 28.7 lbs. per cubic foot. If, as is usually the case, the masonry be heavier than the earthwork, the required thickness of wall would be reduced in

inverse proportion to the square root of the respective weights, so that should the masonry weigh 10 per cent. more than the ballast, the thickness would be about 5 per cent. less than before, or, say, 29.5 per cent. of the height.

For other slopes of repose the equivalent fluid pressure and thickness of wall for materials of equal weight would be as follows:

Ratio of horizontal to vertical	.5	.6	.7	.8	.9	1.0
Fluid pressure...	5.6	7.7	10	12.4	14.8	17.2
Thickness.......	.136	.160	.182	.203	.222	.239

Ratio of horizontal to vertical	1.1	1.2	1.3	1.4	1.5	1.6
Fluid pressure...	19.6	22	24.3	26.5	28.7	30.7
Thickness.......	.256	.27	.284	.297	.31	.32

Ratio of horizontal to vertical	1.7	1.8	2	3	4	∞
Fluid pressure...	32.8	34.6	38.2	52	61	100
Thickness.......	.33	.34	.357	.416	.451	.578

In the thickness tabulated above no allowance has been made for the crushing action on the outer edge; in practice the batter usually given to the face of the wall more than compensates for this action if the mean thickness be that given in the table. No factor of safety is included, but according to theory the wall in each case would be just on the balance. Any one accustomed to deal with works of this class will, however, know that in practice walls so proportioned would in the majority of cases possess a large factor of safety.

Doubtless many engineers will, with the author, have noticed that laborers and others not infrequently carry out unconsciously a number of valuable and

suggestive experiments on The Actual Lateral Pressure of Earthwork. In stacking materials, rough-and-ready retaining walls, made of loose blocks of the same material, are often run up, and as it is generally of little moment whether a slip occurs or not, the workmen do not trouble about factors of safety, but expend the least amount of labor that their every-day experience will justify, and so a tolerably close measure is obtained of the average actual pressure of material retained. When the wood paving was recently laid in Regent Street, the space being limited, the stacked wooden blocks in many cases had to do duty as retaining walls to hold up the broken stone ballast required for the concrete substructure. In one instance (Ex. 1) the author noted that a wall of pitch-pine blocks, 4 feet high and 1 foot thick, sustained the vertical face of a bank of old macadam materials which had been broken up, screened, and tossed against this wall until the bank had attained a height of 3 feet 9 inches, a

width at the top of about 5 feet, and slopes on the farther sides deviating little from 1.2 to 1. Now, referring to the diagram and table of thickness, it will be seen that according to the ordinary theory the thickness of wall which would just balance the thrust of a bank 3 feet 9 inches high of material having a slope of repose of 1.2 to 1 would be $3.75 \times .27 = 1.01$, or, say, 1 foot, which is the actual thickness of the given wall. But in the table the specific weight of the material in the wall and backing is assumed to be the same, whereas in the present case the weight of the pitch-pine block wall, allowing for the height being greater than that of the bank, would only be, say, 46 lbs. $\times \frac{4 \text{ feet}}{3.75 \text{ feet}} = 49$ lbs. per cubic foot, whilst that of the broken granite bank would be, say, 168 lbs. less 40 per cent. for interstices = 101 lbs. per cubic foot. It follows, since the wooden wall stood, that if it had been made of materials having the same weight per cubic foot as the bank, the retaining wall

would not have been on the point of toppling over, as the ordinary theory would indicate, but have possessed a factor of safety of at least $\frac{101 \text{ lbs.}}{49 \text{ lbs.}}$, or, say, 2 to 1. The effective lateral pressure of the earthwork in this instance consequently could not have exceeded a fluid pressure of $\frac{22 \text{ lbs.} \times 49}{101} = 10.7$ lbs. per cubic foot, instead of the 22 lbs., which theoretically corresponds to the given slope of 1.2 to 1.

Taking another case, in which the wall, instead of being lighter than the bank, was much heavier, the same conclusion still holds good. In this instance (Ex. 2) the author found a wall of slag blocks having a batter of $\frac{1}{8}$ of the height, and an effective thickness of 1 foot sustained a bank of broken slag 10 feet high, with a surcharge of some 5 feet more. The battering wall, with a thickness of $\frac{1}{10}$ of the height, would have the same stability as a vertical wall 0.173 thick, and the lateral pressure of the sur-

charged bank with the battering face would be practically the same as that of a horizontal-topped bank with a vertical face; hence, since the relatively closely-packed slag blocks constituting the wall would weigh about 40 per cent. more than the broken slag of the bank, the thickness of a vertical wall built of materials of the same weight as the bank, and having the same stability as the wall under consideration would be $=\sqrt{1.4}\times 0.173=0.205$ of the height. Referring to the table, the figure 0.205 will be found to apply to a slope of repose of 0.8 to 1, whereas the actual slope in the instance of this slag was 1.33 to 1. For the latter slope the thickness theoretically should have been 0.29, and since the stability varies as the square of the thickness, it follows that with the thickness indicated by theory, the wall, instead of being just on the balance, would have possessed a factor of safety of at least $\frac{0.29^2}{0.205^2}$, or 2 to 1, as in the last example.

Other instances of these unintentional experiments on the lateral pressure of earthwork will be found in the stacking of coal in station yards, in the rubbish banks at quarries, and in many other instances which have been investigated by the author, with the invariable result of finding that walls which, according to current theory, would be on the point of failure, really possess a considerable factor of safety.

Turning now from indirect to direct experiments, specially arranged with a view to determine the lateral pressure of earthwork, those carried out at Chatham nearly forty years ago by Lieutenant Hope, R.E., may be referred to. His intention was to experiment first with fine dry sand, as free as possible from the complications introduced by cohesion, irregularities of mass and other practical conditions, and then to extend the investigation to ordinary shingle, and to clay and other soils possessed of great tenacity. Sand and shingle were, however, alone experimented with.

The direct lateral thrust of sand weighing 91 lbs. per cubic foot when lightly thrown together, and 98½ lbs. when well shaken, was measured by balancing the pressure exerted on a board 1 foot square. The mean results of seven experiments (Ex. 3) was 9 lbs. 7 oz., which is that due to a fluid weighing nearly 19 lbs. per cubic foot. As the slope of repose of the sand employed was 1.42 to 1, the theoretical fluid pressure due to the weight of 98½ lbs. per cubic foot would be 26.2 lbs., or about 40 per cent. more than the observed 19 lbs. per cubic foot.

With gravel (Ex. 4) weighing 95½ lbs. per cubic foot, and having a slope of repose of 1⅛ to 1, about the same lateral pressure was found to exist. Lieutenant Hope attempted to reconcile the difference between theoretical and actual results by adding to the measured force an estimated sum for friction against the sides of the apparatus, but experiments of the author's to be subsequently referred to, clearly prove that the difference

is not to be so accounted for. Indeed, the knowledge of what the pressure theoretically should be would appear to have given Lieutenant Hope an unconscious bias in the direction of rather exaggerating the experimental results. This it is extremely easy to do, as a trifling amount of vibration will alter the pressure from 10 to 50 per cent., and a comparatively innocent shake in a small model will correspond in its relative effects with an earthquake in real life.

Experiments with colored sand in a vessel with glass sides did nót uniformly confirm the usual theory that the angle of pressure of maximum thrust is half that contained between the natural slope and the back of the wall (Ex. 5). Thus the line of separation was at an angle of 24° with the vertical instead of 28°. Again, with a gravel bank (Ex. 6) 10 feet high the line of separation ranged from 3 feet 8 inches to 5 feet 8 inches from the back of the wall, whilst as the natural slope was $1\frac{1}{3}$ to 1, the distance should have been 5 feet in all instances if Cou-

lomb's theory applied strictly to even such exceptionally favorable materials as dry sand and shingle.

The really valuable portion of Lieutenant Hope's investigation was the series of experiments on walls built of bricks laid in wet sand. The first of these (Ex. 7) was about 20 feet long and two-and-a-half bricks, or say, 1 foot 11 inches thick. When raised to a height of 8 feet and backed with ballast, it had inclined from the vertical about 1½ inch; at 9 feet the inclination had increased to 3¼ inches, and at 10 feet the wall fell forward in one mass. At the instant when the thrust of the ballast overcame the stability of the wall, the overhang must have been 4 inches, and the moment of stability per lineal foot certainly not more than 2,000 lbs. $\times$ 0.9 foot $=$ 1,800 foot-pounds. Hence, dividing by $\frac{h^3}{6}$, is obtained 10.8 lbs. per cubic foot as the weight of the fluid, which would have exerted a lateral pressure equal to that of the ballast piled against this 10-feet

wall. This is hardly more than half the pressure obtained with the 1-foot square board, and shows how desirable it is that even the most faithful experimenter should not know what to expect if a mere shake of a table will enable him to obtain the desired result. The natural slope of the ballast being 1⅛ to 1, and the weight 95½ lbs. per cubic foot, the pressure theoretically should have been 23.6 lbs. per cubic foot instead of 10.8 lbs.; hence a wall so proportioned as to be on the point of toppling over, according to the ordinary theory, would in this instance have had a factor of safety of rather more than 2 to 1.

Another vertical wall (Ex. 8) was constructed with the same amount of materials differently disposed. At 8 feet high, after heavy rain, the 18-inch thick panel between the 27-inch deep counterforts had bulged 1¼ inch; at 12 feet 10 inches the bulging had increased to 4½ inches, and the overhang at the top to 7½ inches, when, after some hours' gradual movement, the wall fell. The moment

of stability at the time of failure could not have exceeded 2,600 lbs. $\times$ 1 foot = 2,600 foot pounds, which, divided by $\frac{h^2}{6}$, gives 7.4 lbs. per cubic foot, instead of the theoretical 23.6 lbs., as the weight of the equivalent fluid. This result is clearly not evidence that the pressure of the ballast was less in the counterforted wall than in the wall of uniform thickness, but that the binding of the ballast between the counterforts increased the stability of the wall by practically adding somewhat to its weight.

A wall with a batter of $\frac{1}{5}$ of the height, and with counterforts of the same thickness as the last (Ex. 9), was next tried, with noteworthy results. This wall, only 18 inches thick, with counterforts 3 feet 9 inches deep, measuring from the face of the wall, and 10 feet apart, was carried to a height of 21 feet 6 inches without any indications of movement, beyond a bulging about halfway up of $2\frac{1}{2}$ inches at the panel, and $1\frac{1}{2}$ inch at the counterfort; and in Lieutenant Hope's

opinion it would probably have stood for years without giving way any more, although the mean thickness was less than $\frac{1}{10}$ of the height. The calculated stability indicates that a fluid pressure of 8.5 lbs. per cubic foot would have overturned the wall, and, correcting for the reduced thrust of the ballast due to the batter of its face, the equivalent pressure on a vertical wall would be that of a fluid weighing 10 lbs. per cubic foot.

Here, again, doubtless the binding of the gravel between the counterforts contributed to the stability of the wall; but, even adopting the extreme and impossible hypothesis that the ballast was as good as so much brickwork, or, in other words, that the wall was a monolithic structure of the uniform thickness of 3 feet 9 inches, its stability would barely balance the 23.6 lbs. per cubic foot fluid pressure theoretically due to the weight and slope of repose of the backing. Assuming that the binding of the ballast between the counterforts increased the stability, as in Examples 8 and

9, by about 45 per cent., the fluid resistance would be 14.5 lbs. per cubic foot; and, remembering that this wall did not fall, though the bricks were only laid in sand, it is reasonable to infer that this interesting experiment confirms the previous conclusion that a properly built wall in mortar or cement, just balancing the theoretical pressure, would really have had a factor of safety of 2 to 1. Other experiments of Lieutentant Hope's justify this inference, and so do the experiments of General Pasley, also made at Chatham many years ago.

General Pasley experimented with loose dry shingle weighing 89 lbs. per cubic foot, and having a natural slope of 1¼ to 1. His model retaining walls (Ex. 10) were 3 feet long, 26 inches high, of various forms and thickness, and weighed 84 lbs. per cubic foot. The stability of each wall was tried by pulling it over by weights before and after backing it up with shingle, and the difference between the two pulls of course represented the thrust of the shingle. When the thick-

ness of the vertical wall was 8 inches, the stability, without shingle, was equivalent to a pull of 47 lbs. applied at the top of the wall, and with shingle, the pull required to upset it was reduced to 30 lbs. The difference of 17 lbs. represents the thrust of the shingle, and throughout the several hundreds of experiments this appears to have been comprised within the limits of 16 lbs. and 24 lbs. The center of pressure being at $\frac{1}{3}$ of the height of the wall, the mean thrust of 20 lbs. at the top will be equivalent to 60 lbs. at the center of pressure, and the area being 6.5 square feet, and the height 26 inches, the actual lateral pressure of the shingle, as deduced from General Pasley's experiments, is equivalent to that of a fluid weighing 8.5 lbs. per cubic foot, instead of 21 lbs. as theory would indicate.

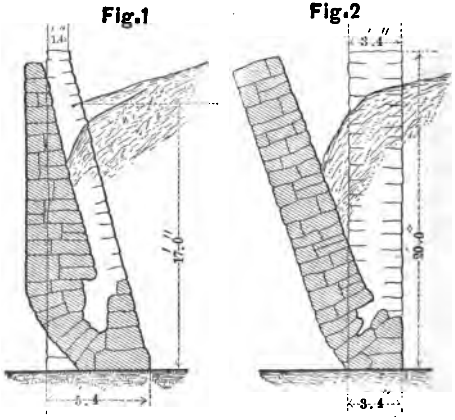
General Cunningham tested some model revetments, and his experiments led him to believe that General Pasley had overestimated the thickness required for stability. The models, in this case

about 30 inches in height, were weighted with earth and musket bullets to the equivalent of an equal mass of masonry weighing 129 lbs. per cubic foot. One of the models (Ex. 11) represented a wall 30 feet high, 6 feet thick at the base, vertical at the back, battering 1 in 10 on the face, with counterforts 4 feet 3 inches thick, 18 feet from center to center, and of a depth equal to the thickness of the wall or, say, 3 feet at the top and 6 feet at the base. This was backed up and surcharged with shingle weighing 104 lbs. per cubic foot, but required a pull of 111 lbs. to overturn it. Another model (Ex. 12) representing a wall 18 feet high, 4 feet 4 inches thick at the base, and 2 feet 8 inches thick at the coping, without counterforts, when surcharged with shingle to a height greater than that of the wall, required a pull of 84 lbs. to upset it. A fluid pressure of 19 lbs. per cubic foot would overcome the stability of such a wall; hence, having regard to the surcharge and to the pull, it will be found that the actual

lateral pressure of the shingle could not have exceeded that due to a fluid weighing 8 lbs. per cubic foot.

General Burgoyne also commenced an experimental investigation of the question of retaining walls, but circumstances precluded his pursuing the subject. About half a century ago he built at Kingstown four experimental walls 20 feet long and 20 feet high, having the same mean thickness of 3 feet 4 inches, or $\frac{1}{6}$ of the height, but differing otherwise. One of them (Ex. 13) was of the uniform thickness of 3 feet 4 inches, and battered $\frac{1}{6}$ of the height; another (Ex. 14) was 1 foot 4 inches thick at the top, and 5 feet 4 inches at the bottom, with a vertical back; the third (Ex. 15, Fig. 1) was of the same dimensions, with a vertical front; and the last (Ex. 16, Fig. 2) was a plain rectangular vertical wall 3 feet 4 inches thick. The masonry consisted simply of rough granite blocks laid dry, and the filling was of loose earth filled in at random, without ramming or other precautions, during a very wet winter.

No. 1 wall stood perfectly, as might have been expected from the behavior of Lieutenant Hope's experimental wall of nearly the same height and batter. No. 2 wall also stood well, coming over only



about 2 inches at the top. A fluid pressure of 22.5 lbs. per cubic foot would be required to overcome the stability of this dry masonry wall weighing 142 lbs. per cubic foot. Earthwork of the class de-

scribed, consolidated during continuous rain, would not weigh less than 112 lbs. per cubic foot, nor have a slope of repose less than $1\frac{1}{2}$ to 1. Referring to the table, the theoretical pressure of such earthwork would be $28.67 \times 1.12 = 32$ lbs. per cubic foot, or nearly one-half greater than the wall could resist.

No. 3 and No. 4 walls both fell when the filling had attained a height of 17 feet. The former came over 10 inches at the top, was greatly convex on the face, overhanging 5 inches in the first 5 feet of its height and rending it in every direction, when finally it burst out at 5 feet 6 inches from the base, and about two-thirds of the upper portion of the wall descended vertically until it reached and crushed into the ground (Fig. 1). The vertical wall tilted over gradually to 18 inches and then broke across, as it were, at about $\frac{1}{4}$ of its height and fell forward (Fig. 2). So long as the wall remained vertical the calculated stability would indicate it to be equal to sustain the pressure of a fluid weighing 20.4 lbs. per

cubic foot, but the overhang of 18 inches and the bulging which occurred would reduce the stability exactly one-half, so that a fluid pressure of 10.2 lbs. would really have sufficed to effect the final overthrow. The character of the failure both of No. 3 and No. 4 walls clearly indicates that if the walls had been in mortar or cement, as usual, the overhang would not have been a fraction of that occurring with the dry stone walling, and the failure would not have taken place. Since, as already stated, the theoretical thrust of the earthwork would be 32lbs. per cubic foot, it is hardly unfair to conclude that a wall in mortar and proportional to that pressure would not have come over and would have enjoyed a factor of safety of at least 2 to 1.

Colonel Michon carried out in 1863 an interesting experiment (Ex. 17) on a 40 feet high retaining wall of a peculiar type (Figs. 3 and 4), which, perhaps, may be best described as a very thin wall with numerous battering buttresses turned upside down. The face wall, battering

1 in 20, was only 1 foot 8 inches thick, and the buttresses, spaced about 5 feet apart from center to center, were also 1

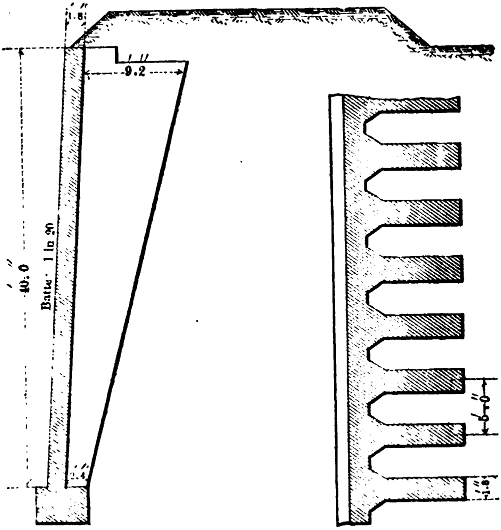


Fig.3   Fig.4

foot 8 inches thick by 2 feet 4 inches deep at the base and 9 feet 2 inches at the top. The work was hurriedly con-

structed during continuous rains with any stones that came to hand, and with very bad lime. When the filling had attained a height of 29 feet the wall bulged a trifle, but no further movement was noticed, though the filling, when carried up to the top of the coping, was allowed some weeks to settle in the rain. Earth was then piled above the level of the coping to a height of between 3 and 4 feet, when the wall fell. The fall was preceded by a general dislocation of the masonry at the base, a bulging at about one-third of the height, and a slight movement of the top towards the bank. The lower portion of the wall fell outwards, the upper part dropped vertically (as in General Burgoyne's wall, Fig. 1), and a considerable number of the counterforts went forward with the slip and even maintained their vertical position.

This failure arose from a flexure of the thin wall at the center of pressure of the earthwork, and would not have occurred had the masonry been in cement

instead of in weak unset lime. No direct data therefor are afforded for an exact estimate of the actual lateral thrust of the heavy wet filling on this lofty wall. Nevertheless, as the weight of the masonry was only 18,000 lbs. per lineal foot, and the center of gravity of the same from the toe but 6 feet 6 inches, it follows that the wall, even if monolithic, would be overturned with the pressure of a fluid weighing 11 lbs. per cubic foot. How far the sodden earthwork between the counterforts contributed to the stability of the wall is open to question, but it could hardly account for the difference between the 11 lbs. or less stability and the 32 lbs. due, according to the ordinary theory, to the weight and slope of the backing. If dirt were as good as masonry, General Burgoyne's wall with the battering back (Fig. 1) would have been more stable than the vertical wall (Fig. 2) in the ratio of the squares of their respective bases, or, say, as 2½ to 1, whereas, these walls proved to be of equal stability, both falling with 17 feet

of filling. Colonel Michon, by assuming dirt to be as good as masonry, and a wall 40 feet high and 1 foot 8 inches thick of unselected stones and unset mortar to be as good as a monolith, succeeds in reconciling the behavior of his wall with the ordinary theory of the stability of earthwork; but in the author's experience the conditions assumed are not approached in practice. The stability of this lofty wall battering only $\frac{1}{20}$ of the height on the face, and averaging hardly more than $\frac{1}{12}$ of the height in thickness, is, nevertheless, one of the most remarkable and interesting facts connected with the subject of the present paper.

To show how invariably an experimentalist is driven to the same conclusion as to the excess in the theoretical estimate of the pressure of earthwork, the "toy" experiments of Mr. Casimer Constable with little wooden bricks and peas for filling may be usually referred to. The peas took a slope of 1.9 to 1, and weighed twice as much per cubic foot as the wooden retaining wall. By

the table the thickness of wall, which would just balance the lateral pressure would be $.35\sqrt{2}=.49$ height. By experiment, a wall (Ex. 18) having a thickness of .40 height moved over slightly, but took some amount of jarring to bring it down. Since the stability varies as the square of the thickness, the calculated wall would be 50 per cent. more stable than the actual wall, without considering the question of jarring. If the slope of the peas had been measured also, after jarring, it would probably have been found to be nearer 2.9 to 1 than 1.9 to 1, and the calculated required thickness would have been correspondingly increased.

The influence of even a slight amount of vibration is well illustrated by the difference between the co-efficient of friction of stones on one another in motion and repose. Granite blocks, which will start on nothing flatter than 1.4 to 1, will continue in motion on an incline of 2.2. to 1, and, for similar reasons, earth-work will assume a flatter slope and ex-

ert a greater lateral pressure under vibration than when at rest. This fact has long received practical recognition from engineers; indeed, attention was called to it by Mr. Charles Hutton Gregory, C. M.G., Past-President Inst. C.E., in a Paper on slips in earthwork, read before the Institution in 1844, when the President and others gave instances of slips in railway cuttings caused by vibration;

The general results of the preceding and other independent experiments on retaining walls tending to throw a doubt on the accuracy of Lieutenant Hope's measurement of the direct lateral thrust of ballast and sand on a board 1 foot square, the author considered it advisable to repeat those experiments. Care was taken to eliminate all disturbing causes tending to vitiate the results. The pressure board was held by a string at its center of pressure, and was perfectly free to move in every direction, which, of course, a retaining wall having a greater hold on the ground than stability to resist overturning has not. In every in-

stance the filling was poured into the bo and allowed to assume its natural slope towards the pressure board, and the latter was rotated and thumped to keep the ballast alive before the reaction was measured. In order to avoid all chance of the bias which the knowledge what to expect might have given him, as it did Lieuten ant Hope, the author had the experiments made by others who were ignorant even of the object of them, whilst he himself purposely experimented with an apparatus the dimensions of which he did not know, and consequently could form no estimate of the weight which would be required in the scale

With clean dry ballast having a natural slope of 1⅛ to 1, it will be remembered Lieutenant Hope obtained a lateral pressure on 1 square foot of 9 lbs. 7 oz. With well-washed wet ballast of the same kind the author found the natural slope to be 1½ to 1, and he decided therefore to use the ballast wet, because, possessing greater fluidity, it would give more uniform results than dry ballast, and also

impose greater lateral pressure. In a large number of independent experiments the results were uniformly as follows (Ex. 19): With 6 lbs. in the scale the board moved forward about ½ inch, but continued to retain the ballast; with 7 lbs. very slight movement occurred; with 8 lbs., no movement at all; and with 10 lbs., under extreme vibration, the board moved forward about as much as it did with 6 lbs. without vibration. The general opinion of the different experiment alists was, that the fair value of the lateral pressure of this wet ballast was 7 lbs., because when that weight was in the scale pad a slight jolt was sufficient to let the ballast down by the run to a slope of 1½ to 1. The board being 1 foot square, this of course is equivalent to the pressure of a fluid weighing 14 lbs. per cubic foot, instead of the 19 lbs. obtained by Lieutenant Hope, and the 26 lbs. indicated by theory.

With the same ballast unwashed and mixed with slightly loamy pit sand (Ex. 20) the natural slope was 1 to 1, without

vibration, and 1¼ to 1 with a moderate amount of vibration. A weight of 3 lbs. was as effective in retaining this ballast as 6 lbs. in the former instance; 4 lbs. held it under a moderate amount of vibration, but 3½ lbs. failed to hold the board under very little. Practically speaking, the lateral thrust was about half that with the clean wet ballast, and considerably less than half that theoretically due to the slope of repose of the loamy ballast.

In harbor works both walls and backing are frequently completely immersed, and, so far as gravity is concerned, stone blocks and rubble become then transformed into coal. The author, therefore, experimented (Ex. 21) with some coal having a peculiarly "greasy" surface, and offering the advantage of exceptional fluidity. With 3 lbs. the board moved forward about 1 inch, but no more until a slight jar was applied, when it fell; with 4 lbs. a moderate amount of vibration also generally caused failure; with 5 lbs. the board usually moved for-

ward gradually without making a rush so long as a tolerably considerable amount of vibration was maintained. When a slip occurred the slope was invariably $1\frac{3}{4}$ to 1. The coal proved to be more sensitive to vibration than the wet ballast, and still more so than the unwashed ballast. Aweight of 4 lbs. with the coal appeared to be equivalent to 7 lbs. with the washed and $3\frac{1}{2}$ lbs. with the unwashed ballast. The weight of the coal being one-half that of the wet ballast, and the respective slopes of repose being $1\frac{3}{4}$ and $1\frac{1}{2}$ to 1, the lateral thrusts would theoretically be as 16.8 : 28.7, which is practically the experimental result of 4 to 7.

The author having occasion to design a solid pier 42 feet in height from the bottom of the harbor to the surface of the quay, where a soft bottom of great thickness and small consistency precluded the use of concrete block or other retaining wall, adopted an arrangement in which an iron grid of rolled joist, with a backing of large blocks of rubble, was substituted for a wall. It was necessary,

therefore, to know the lateral thrust of large blocks of stone in such a structure, and mistrusting theoretical deductions, the author made direct experiments on a model to a scale of 1 inch to the foot.

In this instance the individual stones were intended to be fairly uniform in size, and of lateral dimensions not less than $\frac{1}{20}$ of the height of the wall, so that the conditions differed considerably from those assumed in theoretical investigations. A number of billiard balls exactly superimposed in a tightly fitting box would exert no thrust though their slope of repose might be as flat as 3 to 1; and it is not quite clear how nearly or remotely large boulders in an iron cage approximate to that condition. The stones used in the experiment were waterworn pieces of schistose rock having a "greasy" surface and a slope of repose of $1\frac{1}{2}$ to 1. This inclination was found by the author to obtain in natural slopes of all heights, from the pile of metalling by the roadside to the hills themselves. He ascended one slope of

$1\frac{1}{2}$ to 1, over 500 feet in height, and found the balance was so nearly maintained that a footstep at times would set many tons of stones in motion; and a few winters ago a couple of stones of the respective weights of 18 tons and 22 tons, descended this slope and acquired sufficient momentum to carry them across a road at the foot of the slope and on to the middle of the lawn in front of an adjoining shooting lodge.

The conditions of the material were thus favorable, as in the instance of the coal, for obtaining uniform results and a maximum lateral thrust. In order to exclude all possible influence from side friction, the length of the box was made four times the height of the wall. As the result of ten experiments (Ex. 22) it was found that a weight in the scale corresponding to the pressure of a fluid weighing 10.2 lbs. per cubic foot sufficed to retain the rubble, though the face planking moved forward slightly as the last few shovelfuls were thrown against it. The weight of the stone filling was

98 lbs. per cubic foot, or practically the same as that of the wet ballast last referred to; so the 10.2 lbs., or say under slight vibration 11 lbs., in the present instance compares with the 14 lbs. of the previous instance, and the difference is a measure of the influence of large-sized, smooth-faced boulders as compared with ordinary ballast.

With coal of the same size (Ex. 23) the equivalent weight of fluid was 6 lbs. per cubic foot, which confirms the preceding result when regard is had to the respective weights and slopes of repose of the two materials. The experiments collectively proved that a wall, which according to the ordinary theory would be on the point of being overturned by the thrust of a bank of big boulders, would in fact have a factor of safety of nearly 2½ to 1.

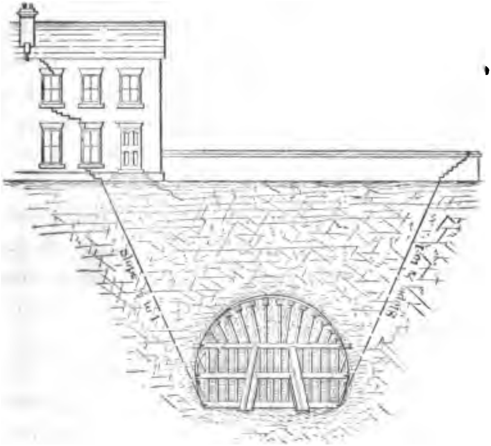
Having thus briefly reviewed some direct experiments on the actual lateral thrust of earthwork, the author proposes to revert to the consideration of indirect experiments, dealing first with a few of

those arising on the 34 miles of deep-timbered trenches and other works of the "underground" railway.

In tunneling very valuable evidence was afforded of the direction of the line of least resistance in a mass of earthwork. From the coming down of the crown bars, the changing of props, the crushing of timber, the compression of green brickwork and other causes, a settlement of from 6 to 8 inches usually occurred overhead, with a general draw of the ground towards the working end of the tunnel and the formation of fissures, attaining a maximum size where the line of least resistance cut the surface. Even when the settlement was slight, fissures were invariably observed in advance of the working end, and in continuous lines running parallel with the tunnel. The slope of these fissures was so uniformly at the angle of ½ to 1, measuring from the bottom of the excavation (Ex. 24, Fig. 5), that the resident engineer professed to be able to foretell with certainty where a building or fence wall,

standing over the tunnel, would crack most. Assuming this $\frac{1}{2}$ to 1 to represent Coulomb's line of least resistance, then the corresponding natural slope of repose

Fig.5



of the material would appear to be $1\frac{1}{8}$ to 1, which is considerably steeper than what it was in fact.

There is nevertheless a closer accord than usual between theory and fact in

the instance of the several miles of fissures, which occurred during the construction of the tunnels of the Metropolitan railway. In other instances, such as the failure of ill-devised timbering, or the pushing forward of a retaining wall, by heavy clay pressing against its lower half, this accord was not always exhibited. In some cases no previous fissures have occurred, but a wedge of 1 to 1 has at once broken off and gone down with the timbering, whilst in others the fissure has appeared immediately at the back of the wall; indeed in one instance, for several consecutive weeks, the author was able to pass a rod, 15 feet long, between the wall and the apparently unsupported vertical face of the ground behind it. The corresponding theoretical slope of repose would thus appear to be horizontal in one case and vertical in the other, which is sufficient evidence of the necessity of giving but a qualified assent to any theoretical deduction affecting the line of least resistance in earthwork.

A very fair notion of the relative in-

tensity of lateral and vertical pressure in earthwork is often obtained in carrying out headings. The heading for the Campden Hill tunnel of the Metropolitan railway is a case in point (Ex. 25). The ground consisted of sand and ballast, heavily charged with water, overlying the clay through which the heading was driven, at a depth of 44 feet from the surface. After the heading had been completed some months, the clay became softened to the consistency of putty by the water which filtered through the numerous fissures, and the full weight of the ground took effect upon the settings. Both caps and side trees showed signs of severe stress throughout the entire length of the heading, and the occasional fractures in the roof and sides indicated that the timbers were proportionately of about the same strength, or rather weakness. The caps were of 14-inch square balks, with a clear span of 8 feet, and the sides of 10-inch square timber, with a clear span of 9 feet. Their respective powers of resistance per square

foot of poling boards, supported, would therefore be as $\frac{14^2}{8^2} : \frac{10^2}{9^2} = 3\frac{1}{2} : 1$.

Now if one thing is settled by experience beyond all question, it is that the superficial beds of London Clay, sodden, as in the present case, with water, will not take a less slope of repose than 3 to 1. The average weight of the wet ground over the heading being about 1 cwt. per cubic foot, the theoretical lateral pressure on the side trees, at a mean depth of 48 feet from the surface, would be (see table) $= 48 \times 0.52 \times 1$ cwt. $=$ 25 cwt. per square foot, and upon the caps$=44 \times 1$ cwt.$=44$ cwt. per square foot, or 1.76 time greater. But the side trees, as has been seen, had only $\frac{1}{3.5}$ of the strength of the caps, so the irresistible conclusion is that the actual lateral pressure of the earthwork in this instance did not exceed one-half of that indicated by theory.*

It is readily shown that the full weight

* See also "Zur Theorie des Erddrucks," Weyrauch Zeitschrift für Baukunde, vol. i., p. 192,

of the ground came upon the settings. Thus, assuming it to do so, the weight upon the caps would be $= 44$ cwt. $\times$ 8 feet clear span $\times$ 3.5 feet distance apart of the settings $= 1{,}232$ cwt., and taking the effective span at 9 feet, the breaking weight, upon the basis of Mr. Lyster's experiments on balks of similar size and quality, would be $\frac{2 \times 14^{2''} \times 2.03 \text{ cwt.}}{9'} =$ 1,240 cwt.; hence the occasional fractures of the balks are fully accounted for. Indeed, the heading would have entirely collapsed, in the course of time, had not the roof been supported by intermediate props practically quadrupling its strength.

In the early days of the construction of the Metropolitan railway, a definite type of timbering had not been arrived at, and some remarkably light systems were tried at times. The lightest the author remembers was the timbering of the 14-feet wide gullet at Baker Street station (Ex. 26). Here the soil cut through was made up of about 8 feet of

yellow clay and gravel, 7 feet of loamy sand, 7 feet of sharp sand and gravel, full of water, and 4 feet of London clay at the bottom of the gullet. The timbering of the lower half consisted of 9-inch by 3-inch walings, 3 feet apart from center to center, in 12-feet lengths, with $\frac{3}{4}$-inch poling boards at the back. With one-half the distributed breaking load, the deflection of this 3-inch deep beam at the span of 12 feet would be at least 4 inches, whilst the ultimate deflection would be measured by feet. As the walings did not bend nearly as much as 4 inches, it will be a liberal estimate to assume that the actual lateral pressure of the earthwork was equal to half the distributed breaking weight of the waling. Having reference to the quality of the timber, this may be estimated at $\frac{3^{2\prime\prime} \times 9^{\prime\prime} \times 2.6 \text{ cwt.}}{12^{\prime}\ 0^{\prime\prime}} = 17.6$ cwt.; and since the area of the poling boards supported by each waling was 36 square feet, it follows that the lateral pressure of the earthwork could not have exceeded 55

lbs. per square foot. But the depth of the bottom waling below the surface was 23 feet, or, neglecting the clay, and taking only the sharp sand and ballast charged with water, the depth would still be 20 feet, and the weight of fluid corresponding to the 55 lbs. per square foot pressure no more than 2.75 lbs. per cubic foot.

It will be remembered that the natural slope of the sand and ballast in Lieutenant Hope's retaining-wall experiments was about 1⅛ to 1, and that the actual and theoretical corresponding fluid pressures were respectively 10.3 lbs. and 23.6 lbs. per cubic foot. In the case of the gullet, the natural slope of the ballast and sand would similarly be not less than 1⅛ to 1, and yet the fluid pressure could not have exceeded 2.75 lbs. This one fact, therefore, is sufficient to prove that the universal assumption of the pressure of earthwork being analogous to that of fluid, and proportional to the depth, is one of convenience rather than truth. The explanation of the singularly small

lateral thrust of the ballast in the present case is to be found in the fact that the ballast was lying between, and partially held back by, the two relatively tenacious layers of loamy sand and clay. As an extreme example of the same kind of action (Ex. 27), the author may state that he once applied to a wooden box full of sand a pressure equivalent to a column of that material 1,400 feet high before the box burst. On the fluid hypothesis, the lateral pressure would have been 1,400 feet $\times$ 23.6 lbs., or about 15 tons per square foot; but of course a few lbs. would have burst the box, and the sand was retained by being jammed between the bottom and lid of the little deal box—the equivalents of the tenacious strata in the gullet.

In shafts the stress on the timbering is far less than in a continuous trench or heading, by reason of the frictional adhesion and tenacity of the adjoining earth. Thus (Ex. 28) at a depth of between 40 and 50 feet, 10-inch square timbers, 4 feet 6 inches apart, proved of

ample strength to support the sides of a 12-feet square shaft, though the same sized timbers at the reduced distance of 3 feet 6 inches apart, failed, as has been seen, to support the sides of a 9-feet square heading.

After the experience of several rather troublesome slips, light timbering was abandoned, and a type which proved to be of ample strength to meet all the contingencies of heavy ground, vibration from road traffic, and the surcharge of lofty buildings, was adopted. In this type (Ex. 29) the 14-feet walings increased in scantling to 12 inches by 7 inches, were spaced 7 feet apart, and strutted at each end and at the center. At one-half the breaking weight the supporting power of the walings would be about $\frac{7^2 \times 12 \times 3 \text{ cwt.} \times 112}{6.5^2 \times 7} = 670$ lbs. per square foot, and as the depth of the excavation was in some instances as much as 36 feet, this would correspond to a fluid pressure of 18.6 lbs per cubic foot. With ground weighing 112 lbs.

per cubic foot, and a slope of repose of 1½ to 1, the theoretical lateral pressure would be 32 lbs. per cubic foot; and when it is remembered that this does not include any allowance for the surcharge due to contiguous buildings, and that the stress on the timber is taken at fully half the breaking weight, it is clear that the average actual lateral pressure of the earthwork must have been less than half that indicated by theory.

On the extensions of the Metropolitan railway, the same type of timbering was adopted, but the walings were generally 9 inches by 4 inches, and spaced 3 feet apart. The supporting power, upon the same basis as in the last instance, would be about 430 lbs. per square foot. In most cases this strength proved to be sufficient, but in a few instances the walings broke, or showed such signs of distress that additional support had to be given. This was the case in some of the deep trenches along the Thames Embankment, where heavy wet silt was traversed. Near Whitehall Stairs (Ex.

30) the trenches were 40 feet deep, so the elastic strength of the timbering was only adequate to the support of a fluid pressure of 10.6 lbs. per cubic foot, or probably but one fourth of that theoretically due to the material; it is therefore no matter for surprise that the walings proved unequal to their work. The stability of the timbering in more moderate depths was, on the other hand, confirmatory of the general deductions drawn from previous examples as to the wide divergence between the actual and calculated thrust of earthwork.

Turning now from the consideration of the temporary works of timbering to the finished and permanent structures on the underground railway, a similar variation in strength will be found to obtain. The lightest retaining wall on the line is that at the Edgware Road station yard (Ex. 31, Figs. 6 and 7). This wall is 23 feet in height from the top of the footing to the ground level, and has a maximum thickness of but 6 feet 3 inches at the base, out of which has to be deducted a

panel 2 feet 6 inches deep. Calculating the moment of stability at the level of the footings and round a point 3 inches back from the face of the pier—which is a sufficient allowance for the crushing action on the brickwork—M = 4.4 feet, × 8,800 lbs. = 38,720 foot pounds per

Fig.6

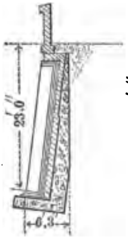
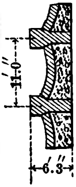


Fig.7



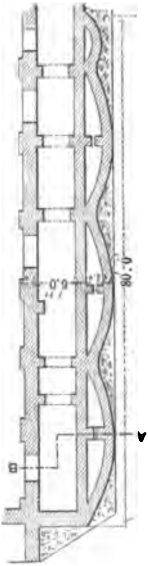
lineal foot of wall. Dividing by $\frac{h^3}{6}$, then 19 lbs. per cubic foot is the weight of the fluid which would overturn this retaining wall. The ground supported is light dry sand, having a slope of repose of about $1\frac{1}{8}$ to 1, and consequently exerting a theoretical lateral thrust equivalent

to a 24-lb. fluid. There is practically no tenacity in the soil, as the author remembers seeing demonstrated on one occasion when a horse and cart, approaching too near the top edge of the slope, broke it away and rolled together to the bottom of the 23-feet cutting. Although theoretically deficient in stability, and subject to heavy vibration from the two minutes train service, the wall has stood perfectly without exhibiting the slightest movement. Upon the basis of the results of actual experiments, and having reference to the character of the soil and other conditions, the factor of safety would appear to be about 2 to 1.

A far lower factor sufficed to secure the temporary stability of the dry areas at the station buildings previous to the erection of the arched roofs (Ex. 32). The arrangement at Sloane Square station is shown in Figs. 8 and 9. The joint stability of the front and back walls is the same as that of a solid rectangular wall having a thickness equal to $(2.4 \times 2.5 + 3.8 \times 5.2)^{\frac{1}{2}} = 5.1$ feet, or

say $\frac{1}{6}$ of the height. A fluid pressure of about 9 lbs. per cubic foot would upset such a wall, so the factor of safety, until

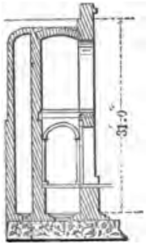
Fig. 8



the arched roof abutted against the dry areas, was only that due to the few 14-inch brick arches which tied the walls together with a certain amount of rigid

ity. This result would perhaps have surprised the author more had he not previously investigated many cases of old timber wharves, in which the piles and planking had lost more than $\frac{3}{4}$ of their original strength from decay, and

Fig.9



Section on line A B.

yet held on against a theoretically over-powering thrust of earthwork.

The relatively strongest wall on the Metropolitan railway system is at the St. John's Wood Road station (Ex. 33), and that has given considerable trouble. Though 8 feet 6 inches thick at the base,

and backed up to a height of 16 feet only out of the total height of 21 feet 6 inches, and supported at the top by the thrust of the arched roof of the station, this wall moved over and forward to an extent which necessitated the immediate adoption of remedial measures.

The moment of stability per lineal foot $M = 73{,}000$ foot pounds, consequently dividing by $\frac{16^3}{6}$ the fluid resistance is 107 lbs. per cubic foot, or allowing for the thrust of the arched roof of the station, considerably greater than that of a perfect fluid having the same density as the ground supported. It is not contended that such a pressure ever occurred upon the wall, although the ground is heavy yellow clay. The failure arose from causes which will be referred to more generally hereafter, and the case is only mentioned as a signal instance of the futility of hoping to reduce the engineering of retaining walls to the form of a mathematical equation.

It is a suggestive fact that, out of the 9 miles of retaining wall on the underground railway, the exceptionally weak wall should show no movement either during or after construction, whilst the exceptionally strong wall, though having six times the stability of the former, should fail. If an engineer has not had some failures with retaining walls, it is merely evidence that his practice has not been sufficiently extensive; for the attempt to guard against every contingency in all instances would lead to ruinous and unjustifiable extravagance, and be indeed as ridiculous a proceding as the making every soft clay cutting at a slope of 10 to 1, because in a few places such cuttings happen to slip down to that slope.

In two instances comparatively heavy retaining walls have failed on the Metropolitan railway. During the construction of the line, the wall on the west side of the Farringdon street station, (Ex. 34), failed bodily by slipping out at the toe and falling backwards on to the

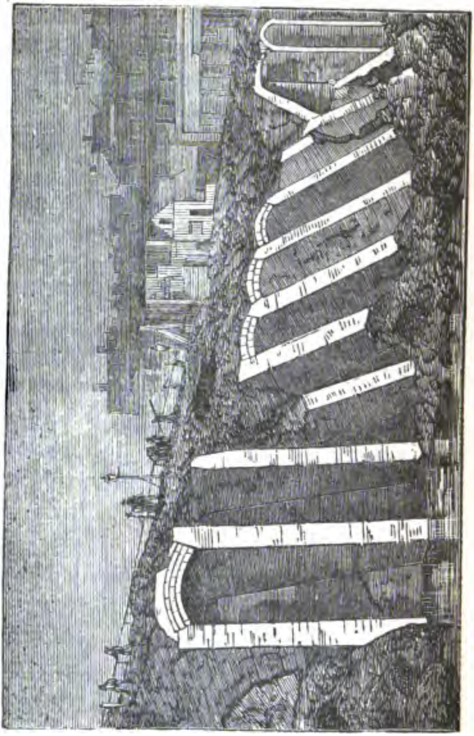


Fig. 10.

slope of the earthwork (Fig. 10). This wall (Figs. 11 and 12) was 29 feet 3 inches high above the footings, and 8 feet

Fig.14

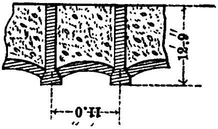


Fig.13

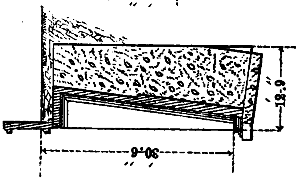


Fig.12

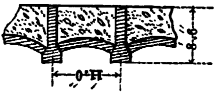
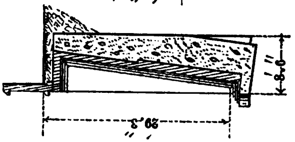


Fig.11



6 inches thick. The ground consisted of about 17 feet of made ground, 3 feet of loamy gravel, and 9 feet of clay. At a distance of 15 feet from the back of

the wall, and at a depth of 15 feet from the surface of the road, was the Fleet Sewer—a badly constructed and much broken brick barrel, 10 feet 6 inches diameter and 3 rings thick. It was believed that the leakage from the sewer induced the failure of the wall, but in reconstruction both wall and sewer were strengthened. The latter was made 4 rings thick in cement, and the former (Ex. 35) was increased in thickness to 12 feet 9 inches (Figs. 13 and 14). Originally the stability was equal to the resistance of a fluid pressure of 24 lbs. per cubic foot, and, as reconstructed, to 54 lbs.

On the opposite side of the same station yard the ground was retained by a line of vaults (Ex. 36, Figs. 15 and 16), 29 feet high above the footings, and 17 feet deep—or double the original thickness of the wall last referred to. Although the resistance to overturning was greater in the proportion of 62 lbs. to 24 lbs. per cubic foot, the vaults some years after construction came over 15 inches

at the top, and slid forward considerably more. The movement when once fairly

Fig. 17

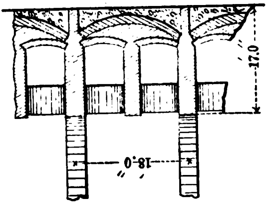


Fig. 16

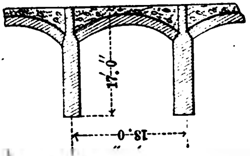
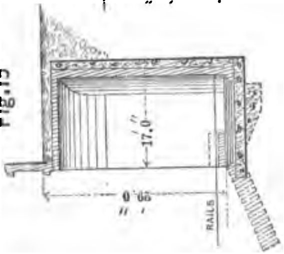


Fig. 15



commenced was rapid and alarming, as a mass of densely inhabited houses was

within 20 feet of the back of the vaults. Steps were promptly taken to strengthen the work, by building intermediate piers and doubling the thickness at the back (Ex. 37, Fig. 17). This arrested the movement for a few months, when the vaults, whose stability had been thus increased to 93 lbs. per cubic foot, again began to go over and slide forward. It was clear that mere weight would not insure stability, so 3-feet square brick struts were carried at intervals from the toe of the piers across and under the railway to the retaining wall of the low-level line traversing the station yard, at a distance of about 34 feet from, and 8 feet below, the level of the footings of the vaults.

The soil in the preceding instance consisted of about 12 feet of made ground overlying the clay, and, as in the former case, a sewer was to be found rather close to the back of the work. Westward of the vaults, the clay encountered in the construction of the line was hard blue clay, requiring the use of a pick,

and portions of the temporary cuttings in the station yard, on the site where the vaults were subsequently built, stood fairly for many months at a slope of 1½ to 1. At one point, however, troublesome slips occurred, and even a 2 to 1 slope had to be piled at the toe to prevent forward movement. It was at this point that the vaults were subsequently found to be most dislocated.

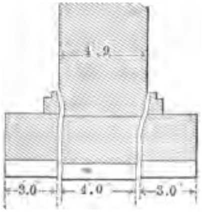
In neither of the above cases was failure due to a deficient moment of stability in the wall, and therefore the fact of their failure does not in any way conflict with the results of the experiments previously set forth. In each case water at the back of the wall was as usual the active agent of mischief—not in thrusting the wall forward by hydrostatic pressure, but in softening the clay and affording a lubricant, so that the resistance was reduced to a sufficient extent to enable the otherwise innocuous lateral pressure of the earthwork to tilt and thrust forward the wall.

A costly, but conclusive, experience of

this softening action was obtained in the instance of the central pier to the double covered way on the District railway near Gloucester Road station (Ex. 38). The weight per lineal foot was 21 tons per foot run of pier, and 4 feet 9 inches was spread out by footings and concrete to a base of 10 feet; hence the pressure on the ground was 2.1 tons per square foot. In a similar construction near Aldersgate the load was 25 tons, and the width of base 8 feet 3 inches, giving the increased load of 2.9 tons upon the foundations. At the Smithfield market the author did not hesitate to place a column carrying 435 tons upon a 12-feet-square base, which is equivalent to a load of 3 tons per square foot; and in the Euston Road, the side wall of the covered way has a load of 15 tons per lineal foot on a 4-feet-wide base, which is at the rate of 3¾ tons per square foot. In all instances the foundation was clay, of apparently equal solidity, and in every instance but the first no settlement at all occurred. For some years no settlement

was observable in that case either, but ultimately, after an accidental flooding of the line, and permanent accumulation of water near the foundations, owing to the line being below the limits of natural drainage, and the pumping being neglected, cracks were observed in the arches, and on examination the concrete

Fig.18



and footings of the central pier were found to be fractured, as shown in Fig. 18. The load of 21 tons per lineal foot was thus imposed upon a base only 4 feet wide, and the softened clay proved unable to sustain the pressure of upwards of 5 tons per square foot. Considerable difficulty was experienced in

checking the movement when once established. The center pier was underpinned with brickwork in cement, but the footings, though of exceptional strength, were again sheared off, and it was found necessary to use 6-inch York landings.

This failure shows the advisability of making concrete foundations of sufficient transverse strength to distribute the weight uniformly over the ground. As the result of experiment, the author is of opinion that the ultimate tensile resistance in a beam of good cement or lias lime concrete, is about 100 lbs. per square inch, and in a beam of good brickwork in cement as much sometimes as 350 lbs. per square inch.* Taking the former value (Ex. 39), a 12-inch thick concrete foundation, projecting 12 inches from the face of a wall, would break with a distributed load $=\frac{12^2 \times 100}{6 \times 6}=4{,}800$ lbs.; or, say, 2 tons per square foot. With a pressure upon the foundation of,

* *Vide* "The Strength of Brickwork." By B. Baker. "Engineering," vol. xiv.

say, 3 tons per square foot, and a factor of safety of 2, the thickness of a concrete foundation would therefore be $\sqrt{\frac{3 \text{ tons} \times 2}{2 \text{ tons}}} = 1.73$ time the amount of its projection beyond the face of the pier or wall, and the author would not advise a less thickness being used when the foundation rests on plastic clay.

Water naturally gravitates to the foundation of a retaining wall, and a softening occurs. Owing to the lateral thrust of the earthwork, the pressure on the foundations is not uniform, and instead of settling uniformly, the outer edge descends fastest and the top of the wall is thrown outwards. The same softening reduces the clay to a condition in which it is easily ploughed up by the advancing wall, and the water acts as an admirable lubricant in diminishing the friction between the bottom of the wall and the clay on which it rests. These elements are exceedingly variable in their nature, and it is practically impossible to

foretell the extent of their influence in each individual case.

In tunneling, clay may be the best or the worst of materials—almost self-supporting, or pressing with irresistible force on the crushing timbers and brickwork. It may be taken for granted that in good ground bad work will occasionally creep into tunneling, however close the inspection. How good a material clay can be was enforced upon the author's attention once in renewing a short length of defective tunnel lining, when on cutting down the work it was found that for some 50 feet the side wall, instead of being 2 feet 6 inches thick as intended, consisted merely of a skin of brickwork 9 in inches thick on the face, with a number of dry bats thrown in loosely behind this thin face wall to fill up the space excavated. This tunnel (Ex. 40) was loaded with a weight of 46 feet of clay over the crown, but no measurable settlement had taken place ten years after completion, and it was rather by sounding the side wall, than by the

observance of cracks, that a suspicion was raised as to its solidity. If the full weight of the ground had come upon the tunnel as it did upon the heading (Ex. 25), the pressure upon the side wall would have been 45 tons per lineal foot, or practically double the strength of the 9-inch work as determined by experiment.

Of course the clay in this case was hard blue clay, which had not been affected by the action of air and moisture. As explained by the Rev. J. C. Clutterbuck, many years ago, the superficial layers of the London clay are yellow, because the protoxide of iron is changed into a peroxide by the action of air and moisture in the disintegrated mass, and it is the yellow clay, therefore, which is the dread of the engineer. As good an example as any of the difference between the two materials was afforded forty years ago, in the well known slip which occurred in the 75-feet-deep cutting at New Cross, when nearly 100,000 tons of yellow clay slipped forward on the hard

smooth surface of the shale—like underlying blue clay, and buried the entire line for a length of more than a hundred yards to a depth of 12 feet.*

Owing to a misunderstanding, a section of concrete wall designed by the author to form one side of a running shed, and to retain the earthwork in a 13-feet cutting through light-made ground, was adopted also in a similar case, but where the ground was heavy wet clay, and the cutting 30 feet deep (Ex. 41). A wall 13 feet in height from formation to coping, and only 3 feet 3 inches thick at the base, had thus to sustain a surcharge of 17 feet. As the slope of repose was at least 1½ to 1, the lateral thrust was theoretically equivalent to a fluid pressure of about 70 lbs. per cubic foot, whereas a pressure of less than one-third that intensity would have overturned the wall. The latter, nevertheless, held up the ground fairly for some months, though the nature of the

* *Vide* Minutes of Proceedings Inst. C.E.. vol. iii., p. 139.

soil was such that it ultimately became necessary to add strong counterforts to the wall, and to reduce the slope of the cuttings generally to 2 : 1.

On the Thames Embankment heavy clay filling was in places cut through by the District railway, and in several instances the light side walls of the covered way were thrust over a few inches at the top before the girders were bedded (Ex. 42). The side walls were eighteen feet in height from the invert to the ground level, and 5 feet 6 inches thick, with panels 5 feet 6 inches wide, by about 2 feet 9 inches deep, and piers 2 feet 6 inches wide. A fluid pressure of 16 lbs. would overcome the stability of these walls, but, though subject to the pressure of the heavy clay filling, none of them failed. The existence of an undue pressure was, however, manifested by the thrusting forward of the green brickwork during the few weeks that the walls were left unsupported by the girders.

The retaining walls, at the approach to Euston Station, afford a good illustra-

tion of the impossibility of making any reasonable approximate estimate of the possible lateral thrust of yellow clay, or of stating positively that no movement will ever occur. These walls, soon after construction, were forced out in an irregular way at the top, bottom, or middle, but on pulling them down, the clay behind appeared to be free from fissures and to stand vertical. Cast-iron struts were subsequently put in between the opposite retaining walls; and although General Burgoyne, who had given much attention to the subject of revetments, prophesied at the time that they would be removed in a few years, "when the ground had become consolidated," the struts still remain, and the walls still give signs of severe and increasing stress.

It is not only London clay that proves so embarrassing to engineers. In a recent Paper* particular attention was called to the treacherous nature of some

* *Vide* "Earthwork Slips," Minutes of Proceedings Inst. C. E., vol. lxiii., p. 280 *et seq.*

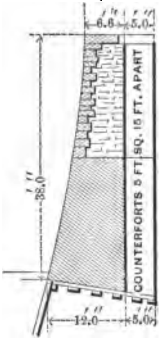
boulder clay which, "although so tough and tenacious as to give the utmost difficulty in excavation, after a short exposure became soft and pasty in the winter, often jolting down the slurry." Examples were given of formidable slips in this material, in contrast with which the author would point to the comparatively slow wasting of the huge boulder clay cliffs near the mouth of the Tyne, a matter which he had occasion to investigate very closely in connection with the Duke of Northumberland's lands in that district. From a comparison of surveys extending over a period of one hundred and fifty years, it appeared that the wasting of the cliff was very slow, and due solely to the wash of the waves at its base. At no time was the slope of repose of this 105-feet-high cliff more than 1 to 1, and in places it stood for years at an average slope of less than $\frac{3}{4}$ to 1. With his experience of North London clay, the author was startled to find people contentedly living in houses partially overhanging the brow of this steep and

ragged cliff, but the stability of the clay was so great, and the wasting so uniform, that the fact of the outhouses being at the bottom of the 100 feet slope, and the main building at the top, did not appear in any way to disturb the equanimity of the householders.

The failures of dock walls, though numerous and instructive, afford no direct evidence as to the actual lateral pressure of earthwork, because in practically every instance the failure is traceable to defective foundations. The author cannot recall any case in which a dock or quay wall founded on rock has overturned or moved forward, though on other foundations a movement to a greater or lesser extent is so much the rule that Voisin Bey, the distinguished engineer-in-chief of the Suez Canal, once stated to the author that he could name no exception to it, since he had failed to find any long line of quay wall, which on close inspection proved to be perfectly straight in line and free from indications of movement. A brief examination of some in-

stances of the failures of dock walls will show how powerfully unknown practical elements affect theoretical deductions in such cases.

Fig. 19



A well-known and often cited case is that of the original Southampton dock wall, constructed now some forty years ago (Ex. 43, Fig. 19). This wall, 38 feet in height from the foundation to the coping, was built on a platform of 6-inch planks, resting on a sandy and loamy bottom. Before the water had been let

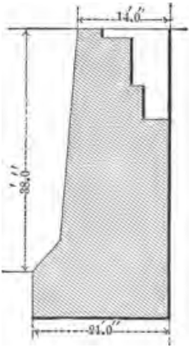
into the dock, or the backing carried to the full height, the wall moved forward in some places as much as three feet, but came over hardly anything at the top. When the water was let into the dock, the filling behind becoming saturated, the pressure on a receding tide exaggerated, and to secure stability it was found necessary to discontinue the filling at some distance below the full height of the wall, and to substitute a timber platform.

The thickuess of this wall at the base is 32 per cent. of the height between the buttresses, 45 per cent. at the buttresses, and a rectangular wall containing the same quantity of material would have a thickness equal to 26 per cent. of the height. Though the base is wide, the weight is light as compared with most other dock walls, and the tendency to slide forward is therefore greater. If founded on a rock bottom, a fluid pressure of about 40 lbs. per cubic foot would have been required to overturn the wall, but of course a fraction of this pressure

would suffice to make it move forward on the actual bottom.

The conclusion drawn by Mr. Giles, M. Inst. C. E., the engineer of the docks, from this and other failures is, that the quality of a dock wall is of little conse-

Fig.20



quence compared with the quantity, and that it ought to be sufficiently strong not only to hold any amount of any kind of backing put against it, but to carry a

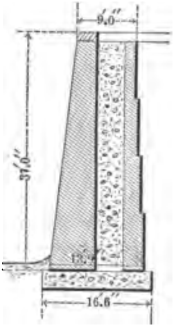
head of water equal to its height if it were left dry on the other side.*

These principles have been adhered to in the recent extension of the Southampton docks (Ex. 44, Fig. 20). Here the wall is founded on a mass of concrete 21 feet wide; the effective thickness at base is about 45 per cent, and the mean thickness 41 per cent. of the height. A fluid pressure of from 60 to 70 lbs. would be required to overturn this wall if on a hard foundation, and probably as much to make it move forward, unless the bottom were of clay or of other unfavorable material. Mr. Giles has found even a heavier wall slide, when founded on a thin layer of gravel overlying clay. In the earlier wall, if the co-efficient of friction of the base on the ground were less than $\frac{2}{3}$, the wall would slide rather than overturn; but in the latter wall, without buttresses, any co-efficient exceeding $\frac{1}{2}$ would be sufficient to prevent sliding.

* *Vide* Minutes of Proceedings Inst. C. E., vol. lv. p. 52.

For comparison with the above, the section of the east quay wall of the Whitehaven dock may be next referred to (Ex. 45, Fig. 21). Having the same height as the Southampton dock wall, the thickness at the base is but 37 per

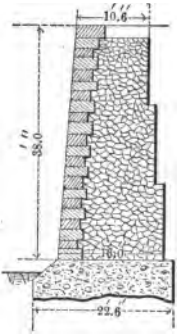
Fig. 21



cent of the height, the mean thickness 31 per cent., and the concrete foundation 16 feet 6 inches, instead of 21 feet wide. This wall has stood perfectly, though it would fail to resist the head of water mentioned by Mr. Giles, but would be

overturned by a fluid weighing from 45 to 50 lbs. per cubic foot. During construction, weep holes were, however, left in the walls to relieve them of hydro-static pressure.

Fig.22



Another dock wall of the same height as the preceding ones, is that of the Avonmouth dock (Ex. 46, Fig. 22). In this instance the thickness is 42 per cent. of the height, and the concrete base 22 feet 6 inches wide, dimensions which,

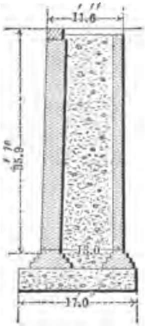
with a good foundation, would enable the wall to stand a full hydrostatic pressure at the back. Owing to the treacherous nature of the bottom, a long length of this wall nevertheless slipped forward at one point as much as 12 feet 6 inches, and sunk 4 feet 6 inches without the latter being affected, whilst at another point, where there was no forward movement. the wall came over about 1 foot 8 inches. When the failure occurred, the foundation rested on apparently stiff blue clay, but in subsequent portions the concrete was carried down through the clay to the sand.* On the east side of the dock, though the walls were founded at an average depth of no less than 9 feet below the bottom of the dock, they still moved forward in the mass some 15 feet 6 inches, and sunk 7 feet 6 inches, The filling was carefully punned in layers, with material which seems to have stood fairly at a slope of 1½ or 2 to 1, so that the wall theoretically possessed an

* *Vide* Minutes of Proceedings Inst. C. E., vol. lv., p. 15.

excess of strength, and yet, owing to the existence of conditions which it was impossible for the engineer to foresee, failures occurred as described.

A somewhat similar case of sliding forward occurred at the New South Dock,

Fig.23



West India Docks (Ex. 47, Fig. 23). The wall is 35 feet 9 inches high from the top of the footings to the coping, and 13 feet, or 36 per cent. of the height, thick at the base. The concrete foundation is 17 feet wide, and 6 feet deep below the

bottom of the dock, and the fluid pressure required for overturning would be about 45 lbs. per cubic foot. A coefficient of friction of less than ½ would be sufficient to guard against sliding under this pressure, but owing to the existence of a thin seam of soft greasy silt between the hard strata of blue clay upon which the foundations rested, several portions of the wall slid forward. The original ground level was about 15 feet below the top of the dock wall, and the excavation stood fairly as a slope of 1 to 1. Favorable material for backing did not appear to be available.

The fact that the stability of a dock wall depends far more upon the foundation than upon the thickness or mass of the wall itself, is well illustrated by the quay wall at Carlingford (Ex. 48, Fig. 24). With a height of no less than 47 feet 6 inches, the thickness of wall and width of foundation at the base are each but 15 feet, or less than 32 per cent. of the height, and the mean thickness is but 24 per cent. A lateral pressure of

half that due to a hydrostatic pressure would probably suffice to overturn this structure.

In contrast with the preceding wall may be cited that of the dock basin at Marseilles (Ex. 49, Fig. 25). In both instances the foundation was good, and the wall rested immediately upon it without the interposition of any broad mass of concrete; but the French engineer, though the wall was but 32 feet high, made the thickness at the base no less than 16 feet 9 inches, or 52 per cent. of the height—an unusually large proportion, which he was led to adopt in consequence of the stratification of the ground inclining towards the wall.

Perhaps one of the boldest and most successful examples of a lightly-proportioned wharf wall is that built by Colonel Michon in 1857 on the Moselle at Toul (Ex. 50, Figs. 26 and 27). With a height of 26 feet, and a batter of 1 in 20, the thickness of the wall through the counterforts is but 3 feet 7 inches at the base, and though the filling is ordinary

Fig.24

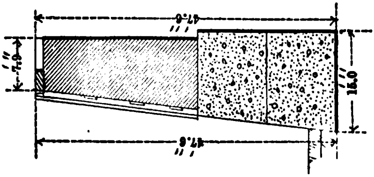


Fig.25

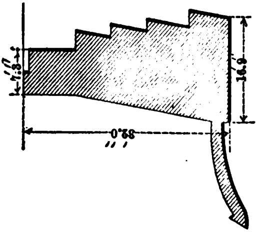


Fig.26

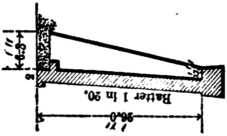


Fig.27

material, having a slope of repose of $1\frac{1}{2}$ to 1, and the floods rise within 6 feet of the top of the coping, no movement whatever has occurred since the wall was built.

As striking a contrast as could be wished to the above light construction is found in Sir John Macneill's quay wall at Grangemouth harbor (Ex. 51, Fig. 28). Both walls are of about the same height, but whilst the mean thickness of the first is only 3.7 feet, or $\frac{1}{7}$ of the height, that of the second, inclusive of the mass of concrete backing, is no less than 23 feet, or, say, $\frac{6}{7}$ of the height.

One of the most troublesome cases of dock-wall failures was that at the Belfast harbor* (Ex. 52, Fig. 29). This wall was founded upon round larch piles 15 feet long, 10 inches in diameter at the top, and 4 feet 6 inches apart from center to center. Symptoms of settlement became apparent soon after the filling was commenced, and some remedial

* *Vide* Minutes of Proceedings Inst. C.E., vol. lv., p. 31.

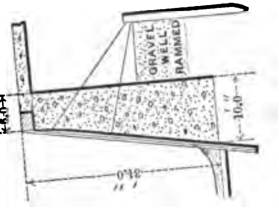


Fig.30

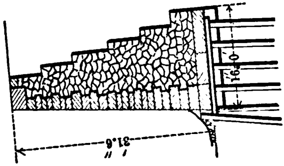


Fig.29

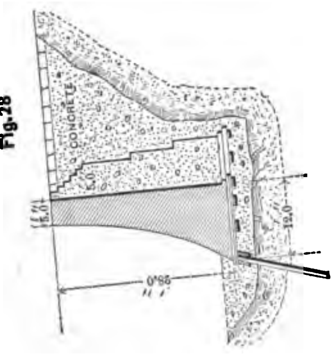


Fig.28

measures were attempted. The ground, however, was hopelessly bad, the slope of repose ranging from 3 to 1 to 6 to 1, and the backing material being equally bad, the light piling was inadequate to resist the thrust. Two years after erection a length of about 70 lineal yards of wall was overturned and carried forward into the middle of the dock entrance, the piles being sheared off about 6 feet below the bottom of the wall. The height from the top of the pile to the coping is 31 feet 6 inches, and the thickness at the base 16 feet, or half the height. On good ground, therefore, the wall would have had an ample margin for stability.

A somewhat similar failure occurred in the instance of the original side walls of the lock chamber of the Victoria docks* (Ex. 53, Fig. 30). These docks were built at a time when little confidence was placed in concrete as a durable material for dock work, and consequently the walls were faced with cast-iron piling and plates, as in previous instances at Black-

* *Ibid.*, vol. xviii., p. 462.

wall and elsewhere. The foundations were on a layer of gravel overlying the clay, but the face piling had little hold in the gravel, and the base of the wall itself was only some 30 per cent. of the height, hence, when the water was let into the dock, the hydrostatic pressure at the back of the lock wall forced it bodily forward into the lock, ploughing up the puddle in front of it, and breaking tie bolts and tie piles as it advanced. In reconstruction a solid concrete wall 20 feet thick, and having nearly treble the stability, was carried through the gravel down to the clay.

The wall of the Victoria Dock Extension Works, by Mr. A. M. Rendel, M. Inst. C.E. (Ex. 54, Fig. 31), has a thickness of about 50 per cent. of the height at the point where the 18-feet wide foundation meets what may be termed the body of the wall, and the wharf wall of Mr. Fowler's Millwall dock (Ex. 55, Fig. 32) has a maximum thickness of 13 feet 6 inches for a height of 28 from bottom of dock to coping, of

practically the same ratio. Either or these walls would be capable of resisting the full hydrostatic pressure.

An early example of a successful wall on a very bad foundation is afforded by

Fig. 31

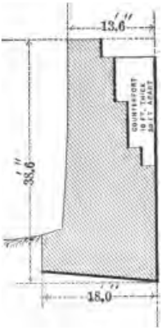
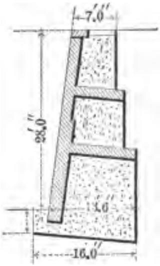


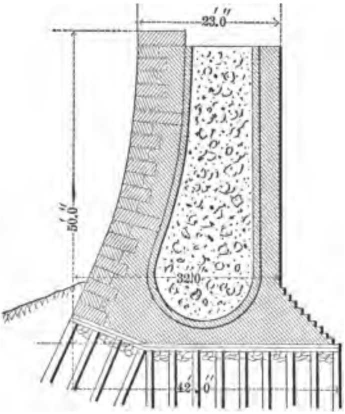
Fig. 32



Sir John Rennie's Sheerness wall (Ex. 56, Fig. 33) The subsoil consisted of loose running silt for a depth of about 50 feet, covered with soft alluvial mud, and the depth at low water was at some points as much as 30 feet. A piled platform about 42 feet in width, with sheet-

ing piles on the river face, and 12-inch piles pitched from 3 to 4 feet apart over the whole area, and driven until a 15-cwt. monkey falling 25 feet did not move

Fig.33



them more than ½ inch at a blow, was prepared, and upon this the wall, no less than 50 feet in extreme height and 32 feet in effective thickness at the base, was raised. In no case has any yielding

or unequal settlement taken place, except in the instance of the basin wall, the cracks in which Sir John Rennie attributed to other causes than a failure in the foundation. Although the voids in the masonry were designedly filled in with grouted chalk and other light material, the Sheerness river wall has perhaps a greater moment of stability than any other wall in the world.

Another exceptionally heavy wall, more than a half century younger than the preceding, is that of the Chatham Dockyard Extension (Ex. 57, Fig. 34). The height from the bottom of the dock to the coping is 39 feet, and the foundations are carried down to the loam gravel or chalk at a depth of 4 feet 6 inches below the bottom of the dock. The thickness of the wall is 21 feet at the base, or, say, ½ of the extreme height. On a hard chalk bottom it would resist a fluid pressure of about 80 lbs per cubic foot.

Two examples of Liverpool dock walls, namely, that at the Canada half-

tide basin, and that at the Herculanean

Fig. 36

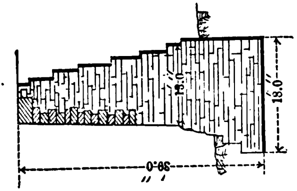


Fig. 35

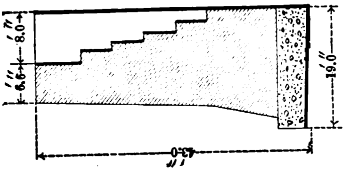
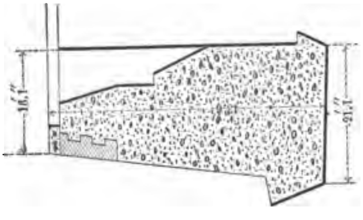


Fig. 34



docks, are given in Figs. 35 and 36. The

former (Ex. 58) is 43 feet in extreme height, and 19 feet, or 44 per cent. wide at the base. The latter (Ex. 59) is 39 feet high, and 18 feet, or 46 per cent. wide at the footings, which rest on a marl bottom. A dock wall at Spezzia (Ex. 60) of somewhat similar proportions, the height being 41 feet, and the width at the bottom of foundations 23 feet, or 56 per cent. of the height is shown on Fig. 37.

Walls made of large concrete blocks, resting upon a mound of rubble, have been constructed in many of the Mediterranean ports, generally with success, but occasionally with failure, as at Smyrna, where, owing to the great settlement, six and seven tiers of blocks had to be superimposed instead of four, as intended, and the quay wall had after all to be supported by a slope of rock in front extending up to within 7 feet of mean sea level, and seriously interfering with the use of the quays. The proportions arrived at by experience are a width of 9 meters at the top, and a thickness

Fig.39

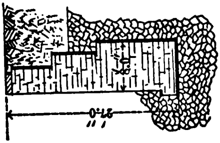


Fig.38

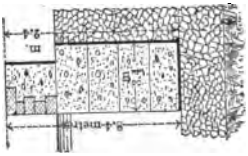
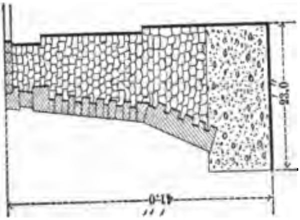


Fig.37



of not less than 2 meters for the rubble mound; a depth of 7 meters below the water line, and a thickness of 4 meters for the concrete block wall resting on the mound; and a minimum thickness of 2.5 meters, and a height of 2.4 meters for the masonry wall coping the concrete blocks.

At Marseilles (Ex. 61, Fig. 38), the top of the rubble mound is only 6 meters below the water-line, so vessels occasionally bump; and the concrete block wall 3.4 meters, or 40 per cent. of the height, in thickness has proved rather less stable under the contingencies of working and the surcharge of buildings and goods than is considered desirable.

Examples are not wanting, however, of walls founded on rubble mounds where the thickness holds a smaller ratio to the height than the 42 per cent., considered necessary by the French engineers. Mr. Fowler has made concrete block walls in the Rosslare Harbor (Ex. 62) 42 per cent. of the height on the sea

face, and but 28 per cent. on the harbor side, but cross walls at 50-feet intervals considerably strengthen the work. The inner wharf wall of the Holyhead new harbor, again (Ex. 63, Fig. 39), is 27 feet high and 8 feet thick, a ratio of under 30 per cent., but though stable, the line of coping is somewhat wavy on plan. The original wall of the West Pier at Whitehaven (Ex. 64, Fig. 40), is 42 feet 6 inches high, with a thickness of 8 feet 6 inches between the buttresses, which latter are 6 feet deep by about 4 feet wide and 15 feet apart; but the lightest of all, perhaps, is the dry masonry outer wall of the St. Katherine's breakwater, Jersey, (Ex. 65, Fig. 41), which is only 14 feet wide at the base for a total height of 50 feet, or a ratio of 28 per cent.

It must not be forgotten, of course, that the three latter walls have to support rubble hearting only, instead of sand and other material, having a much flatter slope of repose. Occasionally, as has been stated (Ex. 22), rubble will not stand at less than 1½ to 1; but at Holy-

head and Alderney the slope of the rubble mound on the harbor side is only about $1\frac{1}{4}$ to 1. At Cherbourg it is 1 to 1, and at Leghorn the large concrete blocks are found to be stable at a slope

Fig.40

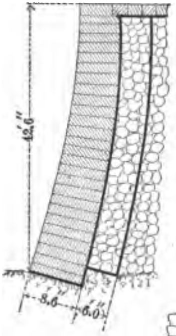
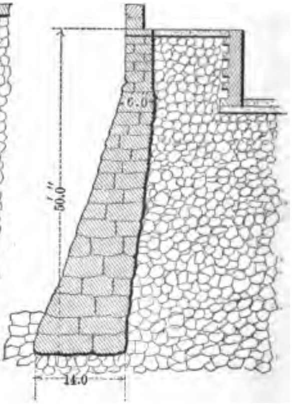


Fig.41



of $\frac{3}{8}$ to 1. By a very little care in selection, the thrust of a rubble filling may be reduced to a fraction of that arising from bad material, and indeed in the ordinary run of fishing piers in the North

of Scotland, however great the height, the face wall of the rubble-hearted pier consists simply of stones from 3 to 4 feet in depth, laid dry to a batter of about 1 in 5. The north-east pier at Seham, again, has an inner wall 25 feet high, battering $1\frac{1}{2}$ inch to the foot, and only 5 feet thick, and many similar examples are to be found at other points of the coast.

The most cursory examination of cases of failure cited above will serve to justify the statement that the numerous dock-wall failures do not afford any direct evidence as to the actual lateral pressure of earthwork. Thus, remembering General Burgoyne's battering wall, only 17 per cent. of the height in thickness, supported the heavy sodden filling at its back, no calculation is required to show that the 32 and 45 per cent. Southampton Dock counterforted wall, the 42 per cent. Avonmouth Dock wall, the 36 per cent. West India Dock wall, the 50 per cent. Belfast Harbor wall, and the 30 per cent. Victoria Dock wall, would all have

stood perfectly had the foundation been rock, as in the instances of General Burgoyne's experimental walls, instead of the mud, clay, and silt which it actually was.

Not only the strength, but the type of cross-section, is singularly indicative of the small influence which theory and experiment have exercised upon the design of dock walls. If the early theorists and experimentalists were in accord upon one point, it was upon the immense advantage afforded by a counterforted wall. Lieutenant Hope was led by his experiments to conclude that if good counterforts were introduced, the merest skin of face wall would suffice for the portion between them, and theorists of course arrived at the same conclusion, from a comparison of the moments of stability of rectangular blocks of masonry edgewise and flatwise. Nevertheless, in only one of the preceding dock walls, and that one forty years old, are counterforts introduced. In practice it was found that counterforts frequently separated from

the body of the wall, and they were consequently regarded as untrustworthy. It is open to question whether this conclusion does not require reconsideration in these days of cheap, strong, and easily-moulded Portland cement concrete. Nothing but blasting would separate the counterforts from a good concrete wall. The author has used concrete in many varieties of structures, and as long back as fifteen years built a four-story warehouse, walls and floors, entirely of concrete, without the introduction of any iron girders. He is bound to admit, however, that by far the boldest and most thorough adaptation of the material to multifarious uses met with by him was in the instance of some farm buildings in an out-of-the-way district in Co. Kerry, Ireland. The small tenant-farmer and his laborers—none of whom were receiving over 11*s*. a week—without skilled assistance of any kind, had constructed dwelling-house, cattle-sheds, and hay-barn wholly of concrete. The cattle-shed was roofed with concrete

arches of 15-feet span, 1 foot rise, and 4 inches thick, springing from octagonal concrete pillars 8 inches in diameter, spaced 15 feet apart from center to center. A layer of concrete constituted the paving, concrete slabs divided the stalls, the cattle fed and drank out of concrete troughs, the windows were glazed in concrete mullions, the gates hung on concrete posts, and the farmer seemed to regret somewhat that he had not adopted concrete doors and concrete five-bar gates.

Portland cement concrete being thus possessed of such great tenacity, there is no risk of counterforts separating from the body of a wall, but it by no means follows that there would be any advantage in using them in other than exceptional cases. In practice, as failures have shown, it is weight, with the consequent grip on the ground, rather than a high moment of stability, that is required in a dock wall. It may be asked, with reason why a bad bottom should affect the thickness of a retaining wall, or, in other

words, why the foundation should not first be made good, and then a wall of ordinary thickness be built upon it. The answer, of course, is that if weight is required to prevent sliding, it is just as economical to distribute the material over the general body of the wall as to confine it to the foundations. It follows, therefore, that under the stated conditions the adoption of a counterforted wall would lead to no economy in material, whilst it would involve additional labor in construction.

A dock wall is subject to far larger contingencies than an ordinary retaining wall, and the required strength will be included only within correspondingly large limits. Hydrostatic pressure alone may more than double or halve the factor of safety in a given wall. Thus, with a well-puddled dock bottom, the subsoil water in the ground at the back of the walls will frequently stand far below the level of the water in the dock, and the hydrostatic pressure may thus wholly neutralize the lateral thrust of the

earth, or even reverse it, as in the case of the inner retaining walls on the Soonkesala canal, some of which, though 35 feet in height, are only 2 feet thick at the top and 7 feet 6 inches at the base. On the other hand, with a porous subsoil at a lock entrance, the back of the walls may be subject, on a receding tide, to the full hydrostatic pressure due to the range of that tide plus the lateral pressure of the filling. Again, the water may stand at the same level on both sides of the wall, but may or may not get underneath it. If the wall is founded on a rock or good clay, there is no more reason why the water should get under the wall than that it should creep through any stratum of a well-constructed masonry or puddle dam, and under those circumstances the presence of the water will increase the stability by diminishing the lateral thrust of the filling. With rubble filling, assuming the weight of the solid stone to be 155 lbs. per cubic foot, and the voids to be 35 per cent., the weight of the filling would be 100 lbs.

per cubic foot in air, and 59 lbs in water, and the lateral thrust will be that due to the latter weight.

If, however, as is perhaps more frequently the case, the wall is founded on a porous stratum, the full hydrostatic pressure will act on the base of the wall, and reduce its stability in practical cases by about one-half. Thus, the 30-ton concrete block walls on rubble mounds, at Marseilles and elsewhere, have the stability due to a weight of, say, 130 lbs. per cubic foot in the air, and 66 lbs. per cubic foot in sea water: but the rubble filling at the back of the wall, being similarly immersed, is also reduced in weight, and consequently thrust to a corresponding extent, so the factor of safety is unaffected.

In walls with offsets at the back, as in Figs. 25 and 36, and water on both sides, the stability will be much increased by the hydrostatic pressure on the top of the offsets, should the wall rest on an impermeable foundation. It is generally

assumed, in theoretical investigations,* that the weight of earthwork super-imposed vertically over the offsets should be included in the weight of the wall in estimating the moment of stability; but the author has found no justification in practice for this assumption. He has invariably observed that when a retaining wall moves by settlement or otherwise, it drops away from the filling, and cavities are formed. A settlement of but $\frac{1}{32}$ of an inch, after the backing had become thoroughly consolidated, would suffice to relieve the offsets of all vertical pressure from the superimposed earth, and the latter cannot therefore be properly considered as contributing to the moment of stability.

A wall with deep offsets at the back is not a desirable form where the foundation is bad, and where, consequently, the pressure over the foundation should be as uniform as possible, so that a settlement may take the form of a uniform

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* *Vide* "A Manual of Civil Engineering." By W. J. M. Rankine, p. 402.

sinking, and not a tilting forward of the coping by reason of the toe sinking faster than the back of the wall. A paneled wall, such as that shown on Figs. 11 to 14, though not admissible in dockwork, is on bad ground far less liable to come over than a wall with offsets at the back, and with a consequent concentration of weight at the front, where the conditions of a lateral thrust especially require that it should not be.

The latter conditions also indicate the expediency of adopting raking piles, as in Fig. 33, rather than vertical piles, as in Fig. 29, where a piled foundation is unavoidable. Thus, taking an ordinary case of dock wall, in which the factor of safety, as regards overturning, is 3, and the ratio of weight of wall to the lateral pressure of earthwork required to overturn it is $1\frac{2}{3}$ to 1, it follows that if the foundation piles are driven at the rate of 1 to $3 + 1\frac{2}{3} = 1:5$ there will be no transverse strain tending to break them off, as in the case illustrated by Fig. 29, and no tendency to plough up the

soft ground in front of the toe of the wall.

If an engineer could tell by inspection the supporting power and frictional adhesion of every bit of soil laid bare, or see through 5 or 10 feet of earth into a "pot hole," or layer of slimy silt, he might avoid many failures, and even hope to frame some useful equations for obtaining the required thickness of a dock wall. Taking things as they are, however, it is hardly worth while to use even a scale and compass in such work, for being in possession of all the information obtainable about the foundation and backing, an engineer may at once sketch as suitable a cross-section for the particular case as he could hope to arrive at after any amount of mathematical investigation. Something must be assumed in any event, and it is far more simple and direct to assume at once the thickness of the wall than to derive the latter from equations based upon a number of uncertain assumptions as to the bearing power of the foundations, the resistance

to gliding, and other elements. This being so, it has often struck the author that the numerous published tables giving the calculated required thicknesses of retaining walls to three places of decimals, stand really on exactly the same scientific basis, and have the same practical value, as the weather forecasts for the year in Old Moore's Almanack. In both cases a pretence is made of foretelling what experience has shown can often not be known until after the event. One well-known authority gives young engineers the choice of five hundred and forty-four different thicknesses for a simple vertical rectangular retaining wall, so that an unfortunate neophyte might not unreasonably conclude that the task before him was not to decide whether, say, a 32-feet wall should be 20 feet thick, as in Example 60, or 9 feet, as in Example 62, but whether it should be 14 feet 6 inches or 14 feet 5½ inches thick.

Although dock wall failures do not afford any data as to the actual lateral

pressure of earthwork, a knowledge of the latter will enable much valuable information to be deduced as to the bearing power of soil and other matters from such failures, and the data so obtained will be applicable to other structures beside retaining walls. Knowing the actual lateral thrust, the coefficient of friction of the base of a wall which has been pushed forward on the ground can be at once deduced, but if the theoretical as distinguished from the actual thrust were introduced into the equation, the result would be valueless.

The aim of the author in the present paper has been to set forth as briefly as possible what he knows regarding the actual lateral thrust of different kinds of soil, in the hope that other engineers would do the same, and that the information asked for by Professor Barlow more than half a century ago may be at last obtained. Although the acquirement of the missing data would probably lead to no modification in the general proportions of retaining structures, since these

are based upon dearly-bought experience, it is none the less desirable that it should be obtained; for an engineer should be able to show why he believes that a given wall will stand or fall. To assume upon theoretical grounds a lateral thrust, which experiments prove to be excessive, and to compensate for this by giving no factor of safety to the wall, is not a scientific mode of procedure.

Experience has shown that a wall $\frac{1}{4}$ of the height in thickness, and battering 1 inch or 2 inches per foot on the face, possesses sufficient stability when the backing and foundation are both favorable. The author, however, would not seek to justify this proportion by assuming the slope of repose to be about 1 to 1, when it is perhaps more nearly $1\frac{1}{2}$ to 1, and a factor of safety to be unnecessary, but would rather say that experiment has shown the actual lateral thrust of good filling to be equivalent to that of a fluid weighing about 10 lbs. per cubic foot, and allowing for variations in the ground, vibration, and contingencies, a

factor of safety of 2, the wall should be able to sustain at least 20 lbs. fluid pressure, which will be the case if $\frac{1}{4}$ of the height in thickness.

It has been similarly proved by experience that under no ordinary conditions of surcharge or heavy backing is it necessary to make a retaining wall on a solid foundation more than double the above, or $\frac{1}{2}$ of the height in thickness. Within these limits the engineer must vary the strength in accordance with the conditions affecting the particular case. Outside these limits the structure ceases to be a retaining wall in the ordinary acceptation of the term. A 9-inch brick facing might secure the face of a friable chalk cutting which, if suffered to remain exposed to the action of the weather, would crumble down to a slope of 1 to 1, and a massive bridge pier, with an "ice-breaker" cutwater, might stand firm against an avalanche, but in neither case could the structure be fairly stated to be a retaining wall.

Hundreds of revetments have been

built by Royal Engineer officers in accordance with General Fanshawe's rule of some fifty years ago, which was to make the thickness of a rectangular brick wall, retaining ordinary material, 24 per cent. of the height for a batter of $\frac{1}{5}$, 25 per cent for $\frac{1}{6}$, 26 per cent. for $\frac{1}{8}$, 27 per cent. for $\frac{1}{10}$, 28 per cent. for $\frac{1}{12}$, 30 per cent. for $\frac{1}{24}$, and 32 per cent. for a vertical wall.

As a result of his own experience the author makes the thickness of retaining walls in ground of an average character equal to $\frac{1}{3}$ of the height from the top of the footings, and if any material is taken out to form a face panel, three-fourths of it are put back in the form of a pilaster. The object of the panel, as of the $1\frac{1}{2}$ inch to the foot batter which he gives to the wall, is not to save material, for this involves loss of weight and grip on the ground, but to effect a better distribution of pressure on the foundation. It may be mentioned that the whole of the walls on the District railway were designed on this basis, and that there has

not been a single instance of settlement, or of coming over or sliding forward. The author has in the present paper analyzed a few dozen experiments, and discussed as many more facts; but an engineer's experience is the outcome not of a few facts, but of the thousands of incidents which force themselves on his attention in carrying out work, and it is this experience, acquired in the construction of works of a somewhat special character, which has convinced the author that the laws governing the lateral pressure of earthwork are not at present satisfactorily formulated.

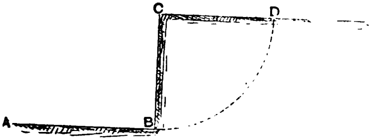
DISCUSSION.

Mr. B. Baker desired to add, that his object in bringing forward the paper was not so much to present certain facts for criticism as to induce others to give the results of their experience, and if every one helped a little he thought a very useful result would be attained.

Mr. W. Airy said he had given consid-

erable thought and attention to the subject of earthwork, and he considered the collection of examples in the paper would make it an extremely useful one for purposes of reference. The subject of earthwork was a very difficult one to deal with, and he wished to point out briefly in what this difficulty consisted. A B C D (Fig. 42) might be taken to be

Fig. 42



the section of some ground having a small vertical cliff at B C. There would be a tendency for the ground to break away and come down along some such line as D B. The whole problem of the stability of the ground, both as affecting the slope of the earth and the pressure against a retaining wall, depended upon the accurate determination of the line D

B. It was not an exceedingly difficult matter to determine this line, if the constants of cohesion, friction, and weight of the ground were known; and he had himself dealt with the problem in a paper communicated to the Institution. The mechanical conditions of equilibrium were very simple; the force tending to bring the earth down was the weight of it; the forces tending to keep it from coming down were the friction along the line D B and the cohesion of the ground along that line. All those forces acted according to well-understood laws, and therefore if the constants of weight, cohesion, and friction of any particular ground were known, it was not difficult to find out the exact position of the line D B, and therefore the pressure on the retaining wall, or the shape of the slope. The question then arose, what was the real difficulty of constructing tables for practical use with regard to earthwork? Simply this, that the varieties of ground were infinite in number and very wide in range, and

when that was the case it was quite idle to think of constructing tables for practical use. A man having a particular kind of earth to prescribe for, would not be able to ascertain by inspection what the constants of that earth were, and therefore he would not know whereabouts in a table to look; he would have to determine the constants for himself; and if he had to do that he had to do the whole work, and the tables were of no use to him. He thought the author had rather overlooked the enormous number of conditions of earth when he contrasted the small number of experiments upon earthwork with the large number of experiments made with timber. A piece of oak would give very nearly the same results for strength, elasticity, and so on, whether it was grown in Kent or in Yorkshire; and, therefore, when a few experiments had been made upon it, it was not necessary to repeat them over and over again. That was not the case with earthwork, because the conditions were so exceed-

ingly variable. He exhibited a little rough machine he had used for testing earthwork and taking the cohesion of the ground. The block of wood might be taken to represent a block of raw clay taken out of a cutting. There was a common lever balance, and a couple of movable cheeks were fitted into chases cut in the sides of the clay block; and the clay having been rammed in a box so that it could not move, weights were put in the scale until the head was torn off. After subtracting the weight of the piece that was torn off, and measuring the area of the cross section that was broken, the constant of cohesion was determined. For the constant of friction he arranged a certain number of blocks of the same clay in a tray, and scraped them off smooth; then he had another block of clay with a smooth surface which he put on it, and then tilted the tray until the loose block slid; that gave the coefficient of friction. He should like to refer to the exceedingly wide range of tenacity shown by different

kinds of clay. In one set of experiments with ordinary brick loam, that clay gave a coefficient of cohesion of 168 lbs. per square foot, and a coefficient of friction of 1.15. With some shaley clay out of a cutting in the Midlands, he had found a coefficient of tenacity of 800 lbs. per square foot, and a coefficient of friction of 0.36. That was a very wide range, but it was only a part of what was actually to be found in practice.

Mr. L. F. Vernon Harcourt wished to say a few words on the subject, as the author had referred to two or three works with which he had been connected. The author had pointed out, from the experiments he had recorded, that the pressure upon the back of a retaining wall was a good deal less than it was theoretically supposed to be—about one-half—but as he allowed a factor of safety of 2, it apparently came to very much the same thing. With regard to walls on a rubble mound, the author remarked that the base was in many cases small. That, he thought, was owing to two

causes; first, that with a rubble mound for a base there was no chance of sliding; and secondly, that in those cases there was a rubble filling behind, which he supposed was about as good a material for backing as could be got. The slope of the inner face of the rubble mound of the breakwater at Alderney harbor had been referred to as $1\frac{1}{4}$ to 1; but it ought to be remembered that in that case the materials used were very large blocks of stone, and therefore the slope would be naturally steeper than under more ordinary conditions. Reference had also been made in the paper to St. Katharine's breakwater, Jersey, as an example of a wall built with a very small base. The author took the whole of the height of that wall as the proper height; but it would be observed that the top of the wall had what used to be called a promenade along it, and therefore the whole of the filling did not apply to the entire height of the wall. The author stated that the base was 28 per cent. of the height of the wall, but

leaving out the promenade it would be 35 per cent. Of course it would be something intermediate, as there would only be the small piece of filling under the promenade to be taken into account additional, instead of what would be the filling at the back if it was filled up entirely to the top level. The author had referred to the West India dock wall, and stated that several portions of it had come forward. That, however, was not quite the case. It was true that two portions of the south wall came forward —that two surfaces of clay at some little depth below the wall slid upon one another. Probably some seam of sand or silt was washed out by the water behind the wall from between two layers of clay, and in that way the two detached surfaces of clay were free to slide upon one another. He was quite certain of the exact position of the surfaces of rupture, because he saw the two surfaces of clay after the excavation was made for rebuilding the wall, and they were as smooth as glass. The remedy for that appeared

to him to be very simple, and it was certainly successful in the case in point. The wall had failed, as the author had stated, not from any fault in the thickness or the weight, but simply owing to the sliding forward; and instead of adding any further weight to the wall, the foundations were carried down to a greater depth; but it only required 2 or 3 feet more in depth in the basin wall foundations that had to be executed afterwards under precisely similar conditions. That was quite sufficient to keep the wall in a perfect state of equilibrium without the least coming forward; and he should imagine that was decidedly better on the whole than adding to the weight of the wall. It appeared to him that practice was rather contrary to theory in giving too great a thickness to the top of the wall, and too small a thickness, comparatively speaking, to the bottom; and that it would be better to have a wall more of the shape of the Sheerness wall a good deal lessened at the top, rather than a wall like those

generally adopted, which had more parallel faces with a little additional thickness from the batter. He thought it would be better to make a wall narrower at the top and widening out more towards the bottom, and to bring the foundations of the wall well down into the ground so as to prevent any chance of sliding. In the case of the West India dock wall, besides the badness of the backing, there was a large amount of water that seemed to percolate from the Millwall docks, which were filled with water while the wall was being built, the docks not having been puddled. It was clearly shown that that had a considerable effect, because the north wall, though it was built in exactly the same manner, and though the water of the Export dock was really nearer, stood perfectly, as there was not the same amount of water pressure at the back, owing to the water being unable to penetrate through the silted-up bottom of the Export dock. He considered that the Institution was much indebted to the author for collect-

ing and comparing so many valuable facts, as, whilst descriptions of particular works were very useful, it was by taking a general survey, from time to time, of the existing state of knowledge, in any special branch, that definite progress in engineering science was most likely to be promoted.

Mr. J. Wolfe Barry believed the statement was true, that the pressure against retaining walls did not approach to the theoretical thrust; at the same time he was of opinion that large retaining walls gave the engineer as much anxiety as any work he ever undertook. It should be remembered that, as a rule, the thrust which the walls had to bear came against them when the material of which they were composed was green, and unless contractors and others were very careful in strutting the new work, and allowing plenty of time for the material to set, there would be a condition of affairs in the early stages of the wall which would never arise after the materials were thoroughly consolidated. He wished to point

out that it was for such reasons most engineers were now getting to realize the extreme desirability of using cement as much as possible. The early stages of engineering works were generally those in which the greatest risks were run, and if a slow-setting material were used, the strains would be exerted against it in its weakest condition, and disasters would occur such as would not happen at a later period. He agreed with the statement of the author with regard to the failure of retaining walls. No doubt, in ninety cases out of a hundred, the failure happened from bad foundations. The remedy in railway works was in many cases that shown in Fig. 17, which practically amounted to strutting the toe of the wall against the opposite wall, and so preventing it sliding forward. That was a very simple arrangement, and resembled in its effect the strutting of timber, which was generally carried out as a temporary measure by a contractor, when, an invert was going to be put in. If the engineer

thought that a continuous invert could be dispensed with, a half measure, which was often perfectly good, was to adopt some of these struts—which, in fact, were a discontinuous invert, as shown in Fig. 17. In railway walls he thought engineers were a little too apt not to use struts above the trains, it being considered in many cases rather *infra dig.* to strut a wall. He could not see why it should be so. The horizontal strains exerted against the retaining wall about 14 feet or 15 feet high above its base were small; the struts consequently involved a very small expense, but they prevented all possibility of movement, and they saved a large amount in the cost of the wall. Having had something to do with the Metropolitan District railway, he could thoroughly corroborate the statement of the author, that the walls had stood remarkably well. As far as he knew, there was no sign of failure or incipient failure in any of them. There was, however, one little matter he had noticed, viz., that in many of the walls

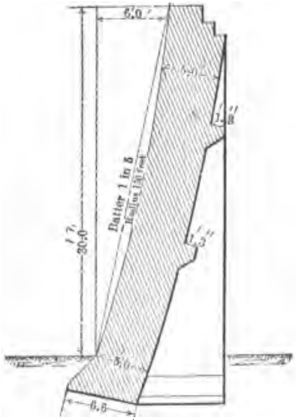
there was a small angular crack across the external angles of the piers. He did not know how the cracks had originated, but he thought they might be due to the action of frost; the corners getting saturated, the frost attacked the brickwork at the angles and broke them off. If so, it rather pointed out that in such walls it might be desirable to round the angles, or have angular bricks and avoid the sharp corners.

Mr. W. B. Lewis said the experience gained in the construction of the Underground railway was so large that the profession naturally looked for the opinions of some of those who were concerned in it; and they all felt grateful for the fullness and ability with which those opinions had been expressed in the paper. He thought the paper was open to this reflection; that, whereas, the author in the earlier pages discredited the theoretical views that generally prevailed respecting retaining walls, in the latter part he stated that his practice had pretty well accorded with them.

For instance, he gave the theoretical thickness for a retaining wall in ground that naturally stood at a slope of $1\frac{1}{2}$ to 1 as 31 per cent. of the height; and in the last paragraph but one he said his habit had been to make his walls $\frac{1}{3}$, or 33 per cent.; and in the Table with slopes from 1 to 1 to 4 to 1, which included all that engineers usually had to deal with, his theoretical thickness ran from 0.239 to 0.451, while in the concluding paragraphs of the paper he stated that the engineer must work between the limits of $\frac{1}{4}$ the thickness and $\frac{1}{2}$, which seemed to agree with the theoretical thickness. The general conclusion that engineers must work between $\frac{1}{4}$ and $\frac{1}{2}$ was different from the practice in which Mr. Lewis had been trained, and he had therefore brought a diagram (Fig. 43) of a retaining wall constructed according to Mr. Brunel's rules. Of course Mr. Brunel, who had to carry out very great works, modified his rules to suit the circumstances; but the diagram represented his standard section of wall such as was

constructed at Lord Hill's land in the early days of the Great Western railway, and at the Britain Ferry docks two years before his death. It would be seen that

Fig. 43



Scale, 16 feet = 1 inch.

the dimensions and peculiarities of that wall differed very much from those given in the paper. In the first place the wall had an average batter of 1 in 5, and at the top a batter of 1 in 10. Batter

was a point on which Mr. Brunel always insisted, and Mr. Lewis was a little surprised that the author seemed to treat it with so much indifference. He was evidently aware of its value, because in the early part of the paper he mentioned a wall with a batter of 1 in 5, and a thickness of 1 foot, which he said was equivalent to a vertical wall of 1 foot 9 inches. Now anything that was equivalent to an increase of the original value of 73 per cent. was well worthy of consideration. Mr. Brunel's custom was to curve the face of the wall. The radius was 150 feet in the case of a 30 feet wall, or five times the height. The thickness was $\frac{1}{5}$ to $\frac{1}{6}$ the height. The counterforts were 2 feet 6 inches thick, and placed 10 feet apart from center to center, but were omitted in good clay cuttings. In the case of docks sometimes there was a difficulty, in consequence of the necessity of having the top more upright, and at Britain Ferry docks the radius was reduced by nearly one-half. Mr. Brunel, too, was in the habit of building

behind what he called sailing courses and the projections in Fig. 43 were 1 foot 3 inches. In the case of embankments the wall was supported by earth carefully punned against it and against the sailing courses, thereby adding considerably to the weight that had to be overturned when pressure came from behind. Then his rule for thickness was $\frac{1}{8}$, which was below the minimum given by the author. There were a number of such walls at Paddington, Bath, Plymouth, Briton Ferry, 30 feet high and 5 feet thick, and generally of nearly the same thickness at the top as at the bottom. Another point Mr. Brunel was particular about was that the footings were made square to the batter, and when the ground was not good considerably larger footings were introduced. At Briton Ferry a 2-feet lining of concrete was employed at some places for watertightness. Concrete was not then in such general use as it was at present. Of course when exceptional ground was met with it was dealt with exceptionally.

At a tunnel on the Wilts, Somerset, and Weymouth railway, some heavy ground had been found; the tunnel mouth was in a 60-feet cutting, a retaining wall 30 feet high was built, and the top was sloped back at ¾ to 1, with a 2-feet covering of masonry, and the wall was built precisely of the dimensions represented by Fig. 43; but as the ground was heavy, the batter, instead of being 1 in 5, was 1 in 4, and that was the only alteration. That wall was built in 1854, had never given any trouble, and was standing at the present moment. It seemed to him that Mr. Brunel, forty years ago, came nearer to the teaching of the experiments and of the reasoning in the paper, than the author had ventured to do in his own practice.

Mr. J. B. Redman observed that the author had undoubtedly filled a void in the literature of engineering; for, notwithstanding the great experience that most of the members of the Institution had of such catastrophes as those which had been referred to, it was only human

like that they had not been often recorded by the designers of the works. Those who constituted what was now a select minority of the Institution would remember the partial failures of Mr. Robert Stephenson's retaining walls in the Euston cutting of what was then the London and Birmingham, and now the London and North-Western railway. Those partial failures were met by overhead horizontal girder struts supporting the walls, and it was rather curious that, notwithstanding all the experience that had been since gained, in a large number of instances, in metropolitan railways, the overhead girder had been, as it were, the natural sequence of what might be termed the unretaining wall. There was one circumstance which very much complicated the question of the direct lateral thrust of earthwork upon a retaining wall, and which rather curiously had not been mentioned by the author. It was incidentally referred to in the latter part of the paper where the author said French engineers, in designing a wall at

Marseilles, made the width of the base 58 per cent. of the vertical height, in consequence of the dip of the strata being towards the wall. In a large number of cases of the failures of retaining walls in open cuttings near London, he thought it would be found that the failure was entirely on one side. Where the dip of the strata was towards the cutting, and more especially if there were laminæ of clay, the superimposed strata often struck near the base of the wall; and a retaining wall on that side not only had to support the normal lateral thrust of the mass of earthwork immediately behind, but it had also a long wedge-like piece of earth impinging against the earth at the back of the wall, so that in many cases the thrust on the wall at the one side must be something like double the amount that it was on the other; because on the other side, the dip being away from the wall, the wall was subject only to the lateral thrust of the earthwork in its rear. The author had stated that the failures of many

dock walls did not illustrate entirely the ordinary lateral thrust of earthwork; but Mr. Redman thought that such cases as the failure of the walls constructed by the late Mr. G. P. Bidder, Past-President Inst. C.E., at the Blackwall entrance to the Victoria docks, the partial failure of the same engineer's walls in the enlargement of the Surrey Docks, the similar catastrophe at the Victoria dock, Hull, in the work designed by the late Mr. John Hartley, and possibly also a similar movement in the South West India Dock wall, were all clearly attributable to lateral thrust. It might be said that the foundation was not taken down deep enough, and consequently the wall did not resist that thrust; but having had a somewhat extended and varied experience for a great number of years, he certainly was not prepared to indorse the dogma that a dock wall or a river wall must necessarily be so strong as to resist a head of water, or in width at the base equal to one-half the height. In the first place, the water ought not to be al-

lowed to come behind the wall. There were exceptional cases, perhaps, where that could hardly be avoided; but it seemed to him that laying down such a tenet was a premium for loose engineering, imperfect supervision, and lavish expenditure. He had himself, in the lower reaches of the Thames, erected some of the heaviest embankment walls on the river, where the thickness was only $\frac{1}{8}$ of the vertical height. It was true that the walls were founded on the best possible foundation—Thames ballast—and it was done as tide work; and the greatest possible care was also taken to keep the backing up to the same level as the wall, and indeed rather above the wall. In fact, the great mistake in retaining walls was the imperfect supervision exercised over the backing. If the backing were put in with tolerably fair material in thin horizontal layers and brought up in that way, the lateral thrust was reduced to a very small matter. The author had stated that the decayed timber wharves on the Thames and

in other neighborhoods showed that the lateral thrust must be over-estimated; but it should be remarked that the skin might be stripped off the face of the earthwork, assuming that no water was coming against it, and it would stand, because from the length of time and consolidation of material, there was no lateral thrust. The example quoted of the breakwater at St. Katherine's, Jersey, appeared to be a case in point. He had nothing to do with the inception or execution of that work; but he thought the wall might be taken down and the heart of the pier would still stand. He would refer to two great Metropolitan failures which were well known, and which might be interesting in illustration of this subject. One was that of Greenwich Pier and the other of the Island Lead Works. The Greenwich Pier was constructed nearly half a century ago from the design of a local architect, Mr. Martyr. It was one of the heaviest embankments on the Thames; it had the greatest depth of water up to

it, and it was, being in the hollow of the reach, subject to every condition of weather. The base was formed by cast-iron piling and cast-iron sheeting between, constituting a half-tide dam, and concrete was got in behind. Upon the top of the concrete there were large 6-inch York landings and a very solid, heavy brick wall. There were also outer piles, and the work was constructed in the best possible way. The case was somewhat complicated by the fact that a large amount of land-water came down and a large amount of spring water. There was a common sewer running through the heart of the work, and a large tidal reservoir for the Ship Hotel. The whole of that work, with the exception of the two returns and quoins and a small portion in front of the Ship Hotel, slipped into the river during the night some forty years back. The late Mr. Chadwick, who built the Hungerford suspension bridge, entered into a contract to restore the work on his own plan, acting as engineer and contractor, and he re-

stored the portion that had failed with timber-bearing piles and a solidly-constructed brick wall. Shortly after the demise of Mr. Chadwick, the restored portion showed signs of failure, and Mr. Redman was called in by the Pier Directorate, and the matter resulted in a lawsuit, and a large sum of money was obtained in compensation. All he did was to bleed the pier by inserting a cast-iron pipe with a self-acting flap at the eastern end, and to remove and substitute with better material some part of the backing. He proposed driving land-tie piles at the back and some in front; but on consideration with the Directorate, it was thought that driving piles might be a ticklish operation. That was twenty years ago, and up to the present time the work had remained in the same state. It had settled somewhat at the eastern end, and there were reopened fissures in front, so that the movement had not altogether ceased. The wall of the Island Lead Works designed by the late Mr. R. Sibley, M. Inst. C.E., was the

pioneer of cast-iron wharfing; and from the fact of the Limehouse cut having been deepened too close up to it, the wall failed. As the author had said, that case did not illustrate the absolute lateral pressure of earthwork, because this work, as long as it was not meddled with, stood satisfactorily. The leaseholders called in Mr. Redman on that occasion, and the freeholder consulted Mr. Bateman, Past-President Inst. C.E., and the late Mr. N. Beardmore, M. Inst. C.E., very wisely—to avoid a lawsuit—constructed a wall deeper down, to their satisfaction.

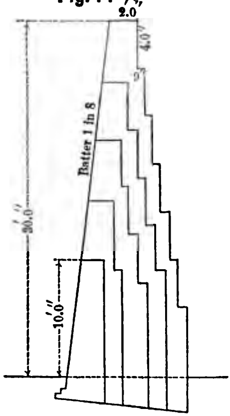
Mr. W. Atkinson agreed with Mr. Lewis's remarks with respect to the large amount of masonry or brickwork that the author had introduced in the cases of the metropolitan railways. He had been much struck with the proportion of $\frac{1}{3}$ of the height for the mean thickness of a wall; but looking at the diagrams, and taking into consideration what he had seen of the work, there was a very good explanation. It struck him

that on the Metropolitan railway, where property was so valuable, the batter which the late Mr. Brunel introduced of 1 in 5 would be extremely inconvenient; either the roadway would have to be narrowed, or a great deal more property would have to be taken, than would be otherwise necessary. No doubt the author would be able to say whether that had any influence in the carrying out of the work. Then with regard to the general question of the walls and their failure due to bad foundations, it struck him that the two things should be entirely separate; that the foundation should be treated as a foundation, and that having been made sufficiently strong, a properly proportioned wall should be placed upon it. He rather gathered from the paper that the two points had been taken as a whole, and that the author meant, "I have a bad foundation, and I will make the whole to stand." If that were so, it would have been better policy to have made a foundation of concrete, and then put a wall sufficiently

strong. With regard to the question of theoretical calculation, there was a French formula which agreed remarkably with what might be considered the ordinary practice. He himself had put up a good many walls, not perhaps as distinct retaining walls, but in connection with bridges on 48 miles of the Mid Wales railway, and he had found practically that the $\frac{1}{45}$ of the height for the mean thickness stood perfectly well. In that case, it was to be borne in mind that there were two elements in addition to the theoretical calculation, namely, the projection of the footings where there was so much leverage, which was not taken into account in the calculation, and the weight of the earth resting on the projections or steppings at the back of the wall, Fig. 44. That, of course, aided the wall very materially; in fact, it might be called so much masonry saved. At all events, if merely the theoretical thickness of the wall was given, then, with the projections of the footings, and the weight of earth on the steppings,

there was a very good margin of safety; and in that way the wall was erected with $\frac{1}{45}$, or 33 per cent. less than the dimension advocated by the author, and

Fig. 44



Scale 16 feet = 1 inch.

was a good and sufficient wall. One point with regard to walls was brought to his notice when in Canada, namely, the thickening of the top to resist frost. In ordinary circumstances the practice

would be to put about 2 feet at the top, and then about 9 feet down a projection of 9 inches, and so on; but in Canada, on account of the penetration of the frost, it had been found necessary to make the top of the wall much thicker than was the practice in England.

Mr. H. LAW desired to add his testimony to the great value of the facts laid before the Institution. It was upon such facts, the result of actual experience, that the most valuable data were formed. In the early part of the paper the author had pointed out that the formula usually adopted—Coulomb's—did not give the results which were obtained when loosely heaped materials were placed at the back of the wall; but a little consideration would show that that formula never was intended to apply to such cases. Coulomb's theorem distinctly took into account the adhesiveness or coherence of the ground, and then determined, depending upon the line on which the ground separated, what the amount of pressure would be; and the value to the

engineer was, that it determined what was the maximum which that pressure could be. Putting w=the weight of a cubic foot of the soil in lbs., h = the height of the wall in feet, r = the limiting angle of resistance of the soil, s = the angle between the line at which the soil separated and the horizontal, and P = the horizontal pressure in lbs. of the soil against the wall, then Coulomb's theorem might be thus expressed:

$$P=\frac{wh^2}{2}.\cot s.\tan(s-r).$$

Now in the case of a fluid, r, or the limiting angle of resistance, vanished, and consequently the result was that the co-tangent of s into the tangent of s became equal to unity, and

$$P=\frac{wh^2}{2}.$$

When the ground was sufficiently coherent to stand vertically, then the angle of separation being 90° the co-tangent of s became nothing, and the pressure became nothing. When the line of

separation coincided with the limiting angle of resistance or r, that was to say, when there was a mass of earth sufficiently coherent not to break of itself, and lying upon a bed which happened to be at the limiting angle of resistance, the tendency of the earth to slide was exactly overcome by its friction, and r being equal to s, the tangent vanished, and P again became nothing. Now, between those two values there was a certain angle at which, if the ground separated, it would produce the maximum pressure, and that was given by Coulomb's theorem, which proved that when the line of separation bisected the angle made by the limiting angle of resistance with the vertical, then $\cot s = \tan (s - r)$, and

$$P = \frac{wh^2}{2} . \cot^2 s,$$

and the maximum pressure was obtained. The great value of the formula was to show, with a given weight of earth and a given limiting angle of resistance, what the maximum pressure was. It could

not exceed the value expressed by making s half the angle between the limiting angle of resistance and the vertical. This formula could not be applied in the case of loose materials, as sand and gravel, because it was impossible for such materials to stand at any other than than their limiting angle of resistance; and under such circumstances there would be upon the wall only a comparatively small pressure, due to the unbalanced weight which remained from the efforts of the sand and the gravel to roll down upon itself. He wished to direct attention to one or two interesting exemplifications of excessive pressure which were met with in the works for the Thames tunnel. The Rotherhithe shaft, 50 feet in diameter, was built upon the surface and sunk by excavating beneath. That operation was successful until a depth of 40 feet was reached, and then, although the exterior surface had been made perfectly smooth by being rendered, it became earth-bound, and notwithstanding the earth was excavated to a depth of 2 feet round

the whole margin, and 50,000 bricks were placed upon the top as a load, making the total weight 1,100 tons, and water was allowed to rise inside, the shaft refused to sink any farther. Now, taking the weight of the ground at 120 lbs. per cubic foot, which was about what it was on the average, and taking the coefficient of friction at 0.67, it would be found that a limiting angle of resistance of about 31° 15′, and a line of fracture of about 27° 30′, would show, by Coulomb's theorem, that the shaft would be bound, and therefore the practical result was quite in accordance with the pressure given by the formula. The author had mentioned a case of some heavy clay which had a pressure equivalent to a fluid pressure of 107 lbs., and if that clay was taken as having a limiting angle of resistance of about 5° or 1 in 10, and the weight was assumed to be 130 lbs. per cubic foot—which clay of that description might very well have—the formula would give 107 lbs. for the fluid pressure. He therefore thought these circumstances

fully showed that where ground was coherent and adhesive, Coulomb's theorem applied. In the progress of the Thames tunnel there had been some remarkable cases of excessive pressure, where of course the weight of the water was superadded to that of the ground. He knew many instances of poling boards, 3 feet in length, 6 inches wide, and 3 inches thick, supported by two poling screws bearing against cast-iron plates, being split lengthwise by the pressure of the earth against the outer surface.

Mr. E. A. Bernays said the inconsistencies alluded to in the paper tended to make it still more interesting than it otherwise would have been. There were few engineers who had carried out works, but were conscious of inconsistencies in their own practice and theories. The author had quoted M. Voisin Bey, the distinguished French engineer, as saying that he had rarely seen a long wall straight, and Mr. Bernays' experience fully confirmed that view. When it was straight the chances were there

was a superabundance of material to keep it so. If it was run fine, as the calculations advised, the chances were 50 to 1 against having a straight wall. With regard to Mr. Brunel's section of wall, no doubt if it had a good foundation it was very strong for the material in it. It not only had a rising abutment to bring the pressure down upon the foundation, but it had counterforts, which added greatly to the strength of the wall, although of late they had gone out of fashion. He considered it was nearer 10 feet at the base than 5 feet, as, if the counterforts were 10 feet apart, the wall was, practically, a solid wall. If made of concrete instead of brickwork, it would probably be found better to make it solid at once. The batter added considerably to the strength, but it was not without practical disadvantages. The greater the batter the greater the disadvantage. The tendency of the batter was to throw the side of a vessel farther away from the wall than need be, and to entail cranes with longer jibs, as well as the use

of much larger fenders. Iron ships were now all covered with anti-fouling composition, which might easily be scraped off. With all its disadvantages he would rather have a smaller batter for practical purposes when ships were to lie alongside the wall. He had seen a wall of this section in Woolwich Dockyard (built, he believed, by Sir John Rennie, Past-President Inst. C.E.), partially pulled down and refaced by the late Mr. James Walker, Past-President Inst. C.E., for the purpose of deepening the dock. It was about 30 feet deep, and was increased to about 38 feet by putting a thin wall in front of it. In pulling down such walls he had always found that the backing in settling hung upon the set off, and he had seen holes under the backing large enough for a man to creep in. He would not say that they were objectionable in other respects, but he preferred a battered back to a retaining wall to square sets off. The author had alluded to a wall that he was building, and had characterized it as "exceptionably heavy." But

for that expression he would have been quite content to sit still: he hoped to be able to show that the exceptionable heaviness was justified by the exceptional circumstances under which it was being built. He did not think much of experiments with peas and pea-gravel, and bits of board a foot square when he had to deal with big walls. The author stated (p. 47) that "experience has shown that a wall ¼ of the height in thickness, and flattering 1 inch or 2 inches per foot on the face, possesses sufficient stability when the backing and foundation are both favorable." Unfortunately for dock engineers it rarely happened that either the foundation or the backing was favorable, and it was still rarer to find both favorable. This fact made the inconsistencies that really showed the thoughtful way in which the paper had been written. There was no attempt to square theory with practice; but the author had candidly pointed out where theory broke down in referring to the retaining walls of the Metropolitan Rail

way, and at the approach to the Euston Station, and in other instances. He agreed with the author that the Sheerness river wall had perhaps a greater moment of stability than any other wall in the world. The section assumed that the pile foundation would stand, though he doubted its stability; but if it would stand, half the thickness of the wall would have been ample. He did not know sufficient of the nature of the subsoil at Sheerness to be able to decide the point. He had been told that in many cases the piles were 40 feet long; and a few years ago, when a new caisson was put in the basin at the yard, there was great fear lest it would come forward when the water was let out. That was merely an instance of the cases where provision must be made for very different calculations from those which were set out in any table. He quite agreed with the author that no calculations would meet cases where the work was exceptionally difficult. In most instances, engineers were called upon to make docks and other great works in the

worst kinds of soils, such as estuaries, beds of rivers, or in deep alluvial deposits. The reason that he strengthened the wall at Chatham was because the original design showed symptoms of weakness, and several of the walls yielded about 10 or 12 inches. He did not say that that was entirely the fault of the walls, because the foundation was far from satisfactory, and there was a decided forward movement of the piles; but it was evident that the wall, for the greater part of the area, was not at any rate too strong for the work. When, however, he came to the east end of the works, where he had to build a wall 1,050 feet long without a single break, and with 35 feet depth of soft mud to excavate through, it was absolutely necessary to strengthen the wall, and it was decided to build it entirely of concrete, in order to be able to give the additional strength without additional cost. He was asked some years ago what angle this mud would assume at rest, and the answer he gave was that it would not lie

flat. The basin at Chatham was being built in an old arm of the river Medway, and the basin generally stood in the middle of the river bed. On each side of it the mud was 35 feet deep on the average, and in some cases the distance to be filled in with backing was 500 feet. The whole of that backing had to be laid on this sliding mud, which brought pressure on the wall in a way far beyond anything he had ever seen allowed for in any calculation. If he understood correctly, Mr Giles had thought it necessary to provide, not only for the backing, but for a pressure of water nearly as high as the water in the dock.. He did not agree with Mr. Redman that it was possible to get the walls built up so as to prevent water percolating. At Chatham there was a standing level of the water in the district, and wherever excavations were made to that depth water was found. The bottom of the basin was 20 or 25 feet below that level, and the water exerted a pressure just as if there was an ocean of that depth behind it. Then it was some-

times necessary to put heavy buildings, as at the Victoria Dock Extension, where large sheds loaded with heavy goods were placed from 100 to 150 feet from the wall. No one would say that such sheds would not exercise a great pressure on the adjacent wall. He would be happy to show the author the wall at the Chatham Dockyard Extension, and abide by that gentleman's judgment, whether "the exceptionally heavy wall" was not neces sary to meet the peculiar conditions of the case.

Mr. A. Giles, after what Mr. Bernays had said about the pressure of mud behind dock walls, thought he was quite justified in adhering to the assertion he made many years ago, that à dock wall ought to be strong enough to carry a head of water behind it equal to its height. He cordially joined in thanking the author for the paper, but he considered it would have been better described as "On the Stability of Retaining Walls." The author had given many examples of dock and retaining walls, but after

throwing over the theoretical calculation as to the pressure of earth against a wall, he said that in ninety-nine cases out of a hundred walls failed from faulty foundations, and not from want of strength in themselves. The various diagrams afforded rather congratulatory evidence of his own theory, that practically all the thick walls had stood, and most of the thin walls had given way. Referring to the old Southampton dock wall, mentioned as having been built 40 years ago, that had only a thickness of 32 per cent. at the base, but with the counterforts it was 35 per cent. That wall had been pushed forward, but it never came down; but it was saved by taking out the wet soil at the top and covering the top by a timber platform. Another wall which he had built had been referred to. That had a thickness of 45 per cent. at the base, and an average thickness of 41 per cent. Surely that wall ought to be strong enough to resist not only the pressure of water behind it, but even the pressure of mud that would not stand at a level.

It had not stood without moving—not from any want of strength in the wall, but simply from the want of adhesion in the foundation. At Whitehaven the thickness of the wall was 37 per cent. at the base, and the mean thickness was 31 per cent., and it had stood. At Avonmouth these values were respectively 59 per cent. and 42 per cent. There was a very fine example at Carlingford of a wall with the base only 32 per cent. thick, and a mean thickness of 24 per cent.; but what could be said about a wall at Sheerness with a height of 40 feet and a base of 43 feet? It was stated that that wall had not moved, and Mr. Bernays had contended that it ought not to move; but he did not think any engineer of the present day would dare to design, or contemplate building, a wall of that character, because a wall of similar height in ordinary ground could be built for £60 a yard, while that wall would cost £300 a yard. In many instances it was not the inherent weakness of the walls that caused them to fall, but the slip at the

bottom, and that was shown in Fig. 17 by the necessity which arose for thickening the wall so as to make the strength as 62 to 24. After all, the wall required still further strengthening by putting buttresses in front of it. The conviction he had arrived at was, that it was not generally the fault of the wall that caused the failure; but the fault of the foundation—not only that the foundation was not wide enough to give sufficient hold on the ground, but that there was not sufficient footing in front of the wall to enable the soil upon which the wall rested to sustain the weight. It was the same as if a cliff, 30 or 40 feet high, were put on tender soil. The soil would not be strong enough to bear it, and consequently the edge of the cliff would settle into the soil, the soil would burst up in front, and the pressure from behind would then make itself felt. He had seen that process take place in a wall which he had constructed, and it was only saved by putting buttresses in front of the footings. Something had been said in the

paper about allowing a margin for contingencies. In that matter every engineer must decide for himself; but he thought that from $\frac{1}{4}$ to $\frac{1}{2}$ was rather a large margin, and he would suggest that the thickness of a retaining wall $\frac{1}{3}$ of the height would be, in nine cases out of ten, ample to resist the backward pressure; but he would insist upon having a large buttress in front of the foundation, carried down as deep as the lowest foundations of the wall. He was at a loss to imagine what the extraordinary projection in front of the wall at Marseilles (Fig. 25) meant. It might be that it was intended to hold the bottom down; and it was in that direction he would recommend retaining walls should be strengthened. A remark had been made about the necessity of having the upper part of dock walls nearly perpendicular, because of the friction of ships rubbing against them, and the inconvenience of ships lying at some distance from the quay at the coping level. That was perfectly true, and he believed it had been a com-

mon practice, in designing walls where there was a curved batter, to make the center of the curve level with the coping, by which a certain depth of almost perpendicular work was obtained from the coping level. He believed that was the correct principle ; but he would urge particularly that, in making dock walls, the foundation should be much wider than they were in general, and that the bulk of the buttress should be in front of the face of the wall, and not behind. In all walls the excavation at the bottom should be carried down perpendicularly, with as little disturbance of the soil as possible; because in excavating the work, it was better to fill up the void so made, that there should be no tendency to slip after the wall was put in. There was another point which he thought was not sufficiently considered by engineers in designing dock walls. They were apt, when the excavation had been carried out, to think that they had got a good foundation ; but he cordially agreed with the author when he used the word "lubricat-

ing." Notwithstanding what Mr. Redman had said, he did not think it was possible to keep water from getting behind a dock wall: he believed there was a point at which water would always be found: it would get up from the bottom or through the wall somewhere; and that being so, he thought that all the soil upon which the wall stood must be soddened and lubricated to a certain extent. He knew of instances where walls had stood for many years; but all at once the moment of lubrication had arrived, and they slipped in. He could only account for it by supposing that there was a tendency on the part of walls to get surrounded with water, by which they became of less specific gravity, or that the soil got saturated, and therefore less able to bear the load put upon it. He would therefore urge upon all his professional brethren who had the conduct of dock works to look particularly to the front of the walls to ensure their stability.

Mr. W. R. Bousfield desired to make one or two remarks, from the theoretical

standpoint which the author had deprecated. He referred to a point in which theory and practice would agree, viz., as to the effect of water behind a retaining wall. If there was an interstice of even an inch the effect on the wall would be exactly the same as if the whole ocean were behind it; therefore, a dock wall should be made to withstand a pressure equal to the hydrostatic pressure due to a head of water of the height of the wall. He wished to ask if the author could explain, somewhat more at length, the effect of lateral pressure in General Burgoyne's experiments, for he did not think the remarks were quite sound. If the lateral pressure of the ground, consisting, say, of loose rubble, was greater than the hydrostatic pressure, the fact of water being admitted would not make the slightest difference, because the water pressed equally on the earth and on the wall in opposite directions, so that the earth would be kept back by the pressure, and the difference between the lateral pressure and the hydrostatic

pressure would be exerted by the earth on the wall. The only effect would be in the distribution of the pressure, which instead of being taken by the points of stone alone, would be distributed by the water over the whole wall. If the lateral pressure of the soil was less than the hydrostatic pressure, then of course, if water was admitted behind, it would exert upon the soil a force greater than the pressure of the soil on the wall; therefore, supposing the soil were rigid, the lateral pressure of the soil would be kept entirely off the wall. Of course, in practice, there were many points at which the pressure was excessive, so that, on the whole, the maximum pressure to be provided for would generally be rather more than the hydrostatic pressure.

Mr. E. Benedict described a retaining wall (Fig. 45) lately put up at Ryde. The ground was sidelong and at the foot of a clay hill, the strata dipping towards the work, and with a heavy building close to it. By cutting a trench and filling it as soon as possible with solid concrete in

Portland cement carried up to the surface, the clay, which weathered rapidly when exposed to the air, was covered

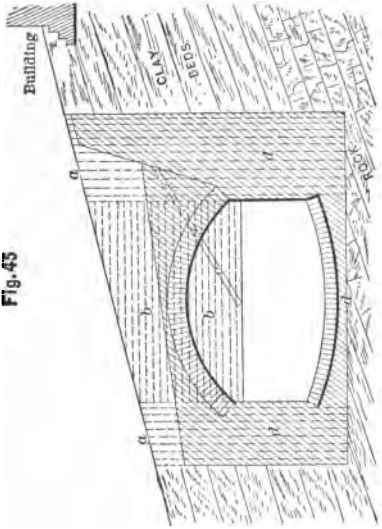


Fig. 45

without delay, the concrete became agglomerated with the clay at the back, and did not allow any percolation of water. The excavation in front of the

wall then proceeded without any movement of the ground occurring, and he thought that none would take place. Eventually a covered way was formed on the lower side of the wall, the arch of which was designed so as to form a continuous lying buttress.

Mr. B. Baker, in reply, observed that he agreed, to some extent, with almost everything that had been said in the discussion, and he considered that the criticism had been very fair. He was glad indeed that he had elicited so many valuable opinions on the subject. Mr. Airy spoke about the difference in the cohesion of different clays. He had noticed the same thing himself, not merely in different clays but in the same clay. A railway cutting often refused to stand at a less slope than 4 to 1, and yet the same clay, after being tempered a little, might be found in an adjoining brick kiln standing with a vertical face. He had nothing to say with regard to Mr. Vernon Harcourt's comments, except that he agreed with almost everything that gentle-

man had said. Mr. Barry had made some sensible remarks about the advantage of using struts to retaining walls, and he thought it would be a good thing, in many cases, to imitate the old architects of cathedrals, and substitute flying buttresses for a heavy mass of materials Some time ago he designed some very cheap sheds upon that principle, in which the roofs were light concrete arches supported by flying buttresses. Mr. Barry had referred to the cracks at the angles of some of the piers of the Metropolitan District railway retaining walls. He was satisfied that these did not arise from pressure, but from chemical action, because they occurred only in the case of certain bricks, and he knew where the bricks came from, and had every reason to mistrust them. He had sometimes found scaling occur all over the face of a wall, though of course the angle was always the weakest point, and nature always tried to round off an angle, as might be seen in Cleopatra's Needle, where there was no square angle.

Mr. Lewis had described a wall designed by the late Mr. Brunel, but he did not approve of it, for reasons that had been set forth by Mr. Bernays. He himself had found exactly the same thing, namely, that in pulling down work where there had been the slightest settlement, the earth at the back did not rest on the offsets, indeed, not infrequently, a man could push in his arm between the offset and the filling. It was therefore idle to maintain, as Professor Rankine and others did, that the earthwork resting on the offset was as good as so much masonry. There was no economy in putting in the offsets, and he attributed the stability of apparently light walls so constructed to the pressure of the counterforts and the good quality of the backing. Mr. Redman had directed attention to the fact that there was an increased thrust when the ground at the back was sloping. No doubt that was so; and a case of that sort was referred to in the discussion on Mr. Constable's paper at the American Society of Engineers,

where the ground at the back was sloping rock. When the wall was first put up and the backing was filled in, the whole mass came forward in consequence of the wedge of earth sliding down the surface of the rock. The masonry was pulled down, the rock cut in steps, and the wall rebuilt of the same thickness. Pig iron to the extent of 55 tons to the lineal foot was then placed behind the wall, and it stood perfectly well, though the thickness was less than 30 per cent. of the height. That was sufficient evidence of the importance of stepping the ground at the back. Mr. Atkinson said he thought it would be better to make the foundation satifactory first and then to build a thin wall on the top of it. At page 43 of the paper that point was alluded to and the answer given. Mr Law had submitted a formula, and drawn deductions from it, which he could not follow; but it seemed to him that the contention was that a loose material exerted less thrust on the wall than a more compact material, and that Coulomb's

theory was not applicable to loose soil. He did not agree with that view in theory, and Lieutenant Hope's experiments showed that that was not so in practice, at least on a small scale. Lieutenant Hope placed a board behind the pressure board at such an angle as to include Coulomb's wedge of maximum thrust between the two boards, and found that the lateral pressure was quite as much when the board was at the slope of repose, $1\frac{1}{2}$ to 1, as when it was at half the angle. There was hardly any difference whether the board was horizontal or at a slope of $\frac{1}{2}$ to 1, or at any intermediate slope. Then it had been remark[illegible] with regard to one of his examples, i[illegible] con[illegible]ch the stability of the wall was equ[illegible]unter-fluid pressure of 107 lbs. per c[illegible] back-that theory would indicate th[illegible]tention to be about that amount; b[illegible]creased ment in the paper did not s[illegible]back was been noticed, that the wal[illegible]so; and a that pressure on it, but fail[illegible] to in the forward. Of course it m[illegible]'s paper at forward with a pressure [illegible] Engineers,

cubic foot, but since the struts had been put in, there was not the slightest indication of movement, and therefore the moment of stability could not have been deficient. Mr. Bernays seemed to imagine that the expression "excessively heavy" reflected on the design of the Chatham dock wall, but the intention was the reverse. He entirely approved of it, and considered that it was a well designed and creditable engineering work in every respect. It was one that he should imitate. His contention throughout the paper was that formulæ did not apply to such works, and although he began the paper with a diagram he set it up merely in order to knock it over. Mr. Giles considered that instead of the limits of $\frac{1}{4}$ to $\frac{1}{2}$ of the height for the thickness of a retaining wall, $\frac{1}{8}$ should be the limit with a buttress in front of the toe; but he did not think that that was the practice which had been followed in the Southampton Dock Extension, where the limit, he believed, was nearer $\frac{1}{2}$ than $\frac{1}{8}$—45 per

cent. The curious slope projection in Fig. 25 was really an apron to protect the foundation, which was of clay. The clay was very hard when laid bare, and a sort of shield was put there to prevent its softening. He believed the same thing had been recommended by a committee of engineers in the case of the Belfast dock, where the wall failed; but it was applied too late, or the conditions were different, because the wall came forward notwithstanding.

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